ADAPTIVE RESERVOIR OPERATION STRATEGIES UNDER CHANGING BOUNDARY CONDITIONS – THE CASE OF ASWAN HIGH DAM RESERVOIR

Submitted to Department of Civil Engineering and Geodesy
For the Degree of Doctor of Philosophy

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M.Sc. in Civil Engineering

A thesis
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ADAPTIVE TALSPERRENSTEUERUNG UNTER VERAENDERLICHEN RANDBEDINGUNGEN - DAS FALLBEISPIEL ASWANSTAUDAMM

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STATEMENT

This thesis is submitted to Darmstadt University of Technology for the degree of Doctor of Philosophy in Civil Engineering.

No part of this thesis has been submitted for a degree or a qualification at any other University or institution.

The work included in this thesis was carried out in Section of Engineering Hydrology and Water Management - Institute of Hydraulic and Water Resources Engineering - Department of Civil Engineering and Geodesy - Darmstadt University of Technology- Germany, under supervision of **Prof. Dr.-Ing. Manfred Ostrowski** and **Prof. Dr.-Ing. Peter Rutschmann**.

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ABSTRACT

ADAPTIVE RESERVOIR OPERATION STRATEGIES UNDER CHANGING BOUNDARY CONDITIONS – THE CASE OF ASWAN HIGH DAM RESERVOIR

During the lifetime of a reservoir its boundary conditions are continuously changing. Such changes include global changes such as climate and economic change as well as national, regional and local changes. National changes are often induced by modified political objectives, regional changes might include supply and demand alterations, while local changes include modifications directly linked to the reservoir infrastructure itself, e.g. reduced storage volume due to sedimentation or dam enlargement or other technical modifications allowing or forcing modified operation strategies and rules.

It must be the objective to be prepared for such changes by analysing the consequences of potential future changes, defined by a set a feasible scenarios (multiple futures). If the consequences of development scenarios are analysed before the changes occur, immediate adequate reactions become possible.

The assessment of sustainable future development includes multiple criteria chosen from the economic, social and ecologic sectors. Adequate political, administrative and technical measures have to be taken to foster sustainable development.

In this thesis BlueM, a model developed by the Institute for Hydraulic and Water Resources Engineering, Section for Engineering Hydrology and Water Management of the Darmstadt University of Technology, Germany, will be used to analyse future development of water resources yield and demand and related modifications of the infrastructure for the case of the Aswan high dam reservoir. High emphasis will be given to the technical and economic aspects of the problem without neglecting the importance and influence of the other sectors.

Analysis will be based on the assumption of regional and local change scenarios, their model based analysis and the proposal of adequate reactions by identifying adaptive reservoir operation strategies.

ABSTRAKT

ADAPTIVE TALSPERRENSTEUERUNG UNTER VERAENDERLICHEN RANDBEDINGUNGEN - DAS FALLBEISPIEL ASWANSTAUDAMM

Waehrend der Lebensdauer eines Staudammes koennen sich die Randbedingungen jederzeit aendern. Zu diesen veraenderlichen Randbedingungen zaehlen globale Veraenderungen wie der Klimawandel oder oekonomische Veraenderungen sowie nationale, regionale oder lokale Aenderungen. Nationale Veraenderungen werden haeufig durch politische Entscheidungen ausgeloest. Zu regionalen Aenderungen zaehlen z.B. der Wasserbedarf oder das verfuegbare Abfluss Volumen. Lokale Veraenderungen stehen in direkter Verbindung mit Aenderungen an der Bauwerksstruktur, z.B. reduzierte Speichervolumina durch Sedimentation, Dammvergroe erungen oder andere technische Ma nahmen, die einen veraenderten Betrieb des Staudammes erfordern oder erlauben.

Zielsetzung ist es, auf die moeglichen Veraenderungen vorbereitet zu sein. Hierzu koennen die Auswirkungen von zukuenftigen Veraenderungen mittels Szenarien analysiert werden. Auf diesem Wege kann beim Eintritt der Veraenderung direkt und adaequat reagiert werden.

Die Beurteilung der nachhaltigen Entwicklung unter veraenderten Randbedingungen erfolgt mit mehreren Kriterien aus dem wirtschaftlichen, sozialen und oekologischen Bereich. Entsprechende politische, adminstrative und technische Ma nahmen muessen ergriffen werden, um eine nachhaltige Entwicklung zu unterstuetzen.

Im Rahme der Arbeit kommt das Modell BlueM zur Analyse veraenderter Wasserverfuegbarkeitsmengen, veraendertem Wasserverbrauch und entsprechenden Veraenderungen der Bauwerksstruktur sowie der Betriebsregeln zum Einsatz. Ein besonderer Schwerpunkt wird auf die technischen und wirtschafltichen Aspekte der Fragestellung gelegt, es werden aber auch die uebrigen Bereiche betrachtet.

Ausgehend von angenommen Szenarien fuer regionale und lokale Veraenderugnen werden diese modelltechnisch untersucht. Basierend auf der Analyse der Modellergebnisse werden Anpassungstrategien fuer den adaptiven Talsperrenbetrieb abgeleitet.

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LIST OF ABBREVIATION & ACRONYMS

The following symbols and notations are used in this thesis:

AHD	: Aswan High Dam.
AHDR	: Aswan High Dam Reservoir.
BCM	: Billion Cubic Meter.
CUR	: Current operation rule.
GCM	: General Circulation Model.
GFDL	: Geophysical Fluid Dynamics Laboratory, USA.
GISS	: Goddard Institute for Space Studies, USA.
Ha	: Hectare.
ICID	: International Commission on Irrigation and Drainage.
KWh	: Kilo Watt-hour.
LEGOS	: Laboratorie d'Etudes en Geophysique et Oceanographie Spatiales (France).
m.a.s.l.	: Meter above sea level.
MCM	: Million Cubic Meter.
MW	: Mega Watt.
MEE	: Ministry of Electricity and Energy, Egypt.
MWRI	: Ministry of Water Resources and Irrigation, Egypt.
NBCBN	: Nile Basin Capacity Building Network.
NWRP	: National Water Resources Plan for Egypt.
OPT	: Optimal operation rule.
ppm	: Parts per million.
TDS	: Total Dissolved Solids.
UKMO	: The United Kingdom Meteorological Office.

1 INTRODUCTION

1.2 BACKGROUND

The growing population of Egypt and related industrial and agricultural activities has increased the demand for water to a level that reaches the limits of the available supply [Attia, 2007]. The population of Egypt has been growing in the last years from a mere 38 million in 1977 to 66 million in 2002 and is expected to grow to 83 million by 2017 [MWRI, 2005]. The present population of Egypt is strongly concentrated in the Nile Valley and the Delta: 97% of the population lives on 4% of the land of Egypt. To relieve the pressure on the Nile Valley and Delta, the Egyptian government has embarked on an ambitious program to increase the inhabited area in Egypt by means of horizontal expansion projects in agriculture and the creation of new industrial areas and cities in the desert. All these developments require water. Egypt has only one main source of fresh water supply, the Nile River, which supplies over 95% of the country water needs. However, the water availability from the Nile River is not increasing and possibilities for additional supply are very limited [MWRI, 2005].

Figure 1.1. Location of Aswan High Dam Reservoir.

There are some winter rains in the delta and along the Mediterranean coast, west of the delta. Non-conventional water resources in Egypt are very limited and often with local importance. They include desalination of 0.025 BCM/year in the tourist areas along the Red Sea and the Mediterranean, wastewater treatment of 0.2 BCM/year for agriculture near Cairo, as of year 2000 [Aquastat, 2005].

They include also flash flood harvesting schemes along the Mediterranean and Sinai [Attia, 2007]. Non-renewable underground fossil water supplies are accessible outside the river valley, especially in the oases. Consequently, agricultural development is closely linked to the Nile River and its management [MWRI, 2005]. The hydrology of Egypt is dominated by the Nile River and its regulation by the Aswan High Dam (AHD).

Construction of the AHD on the River Nile in southern Egypt began in 1960 and was completed on 1972. The dam is in fact the core of all production in Egypt. It is the foundation upon which the country's contemporary industrial, agricultural and economic revival depends. With regard to its relative economic importance, the dam project has a unique position among the big irrigation projects in the world [Volker and Henry, 1997].

Aswan High Dam Reservoir (AHDR), known as Lake Nasser, is a reservoir formed as a result of the construction of the AHD. It is located on the border between Egypt and Sudan. The reservoir has a large annual carry-over capacity of 168.90 BCM [Whittington and Guariso, 1983]. Due to the enormous importance of the reservoir special and national consideration must be given to the reservoir operation and development. Figure 1.1 shows a location map of the reservoir, which represents one of the world's largest artificial lakes [MWRI, 2005].

1.3OBJECTIVES OF THE STUDY

Operation of the AHDR might face different challenges in the 21st century due to potential changes of the demand-supply conditions.

Egypt's water demand might rapidly increase due to the population growth and the improvement of living standards as well as to achieve the government policies in order to reclaim new lands and to encourage development in the industrial sector. The major water consuming sectors are agriculture, municipalities and industries. On the other hand yield supply related of rainfall and evaporation and subsequent changes of inflow into the reservoir must be taken into consideration. These supply scenarios are stochastic and vary year by year. Drivers are global warming and related climate change which will determine these variables. The natural Nile flows are very sensitive to relatively small changes in rainfall [WL, 2004].

Future scenarios are uncertain by definition. Egypt aims to support strongly the socio-economic development, e.g. by providing its inhabitants with access to sufficient drinking water of good quality, by providing water to farmers to irrigate their lands and to industry. But how many people will be there in the future, how much land will need to be irrigated and how much water industry may need? And what about climate change? Will the Nile provide more or less water in the future to be distributed among the riparian countries? All these uncertainties are captured in scenarios. By developing alternative scenarios multiple possible futures can be determined and analysed, trying to find the best strategy to deal with that future and the uncertainties involved. If the consequences of potential development scenarios are analysed before the changes actually occur, immediate adequate reactions become possible.

The main issue of this thesis to investigate potential modification of the reservoir operation strategies for the AHDR. A flexible model (BlueM) will be used to analyse future development of water resources yield and demand and related modifications of the infrastructure and operation rules for the reservoir. Analysis will be based on the assumption of regional and local change scenarios, their model based analysis and the proposal of adequate reactions by identifying adaptive reservoir operation strategies.

1.4 CONTENT AND STRUCTURE OF THE THESIS

This thesis consists of nine Chapters. Chapter 1 is an introductory chapter outlining the problem statement and the objectives of the research work, the scope of the study is clearly stated in this chapter as well as a structure of the thesis.

Chapter 2 presents some of the main articles, studies and researches that were needed for this research. This Chapter also includes general background about the water problem in Egypt, this section is divided to three main parts, the first part is about the Nile basin which consider the main source of water supply to Egypt and the impact of climate change on the Nile inflows, the second part describes the technical and ecological impacts of the AHD, Egypt's water supply and demands in the present and the future are represented in the third part.

Chapter 3 shows the different elements for the multipurpose reservoir system and the general mathematical formulation for the proposed method to modeling the AHDR, in this Chapter also a model is developed and calibrated on the basis of knowledge of the system as an integrated model.

Chapter 4 identifies the data and the processes involved linking the data to build the model. Chapter 5 builds possible scenarios to run the model and compute the results. In Chapter 6 the future hydrologic scenarios developed have been used to assess the expected impacts to potential climate change and basin development scenarios.

In Chapter 7 a dynamic operating rule was devised, the problem of multi-objective optimisation is reviewed to provide basic concepts for solving a multi-purpose reservoir operation problem. A comparison is made of existing operating policy for the AHD with that resulting from a dynamic operating policy. Chapter 8 identifies the level variation in the AHDR using satellite altimetry data, and evaluates impact of level variation on the reservoir operation. The conclusions obtained from the study and future works are presented in Chapter 9.

2 LITERATURE REVIEW

2.1 ADAPTIVE MANAGEMENT

Adaptive management can more generally be defined as a systematic process for improving management policies and practices by learning from the outcomes of management strategies that have already been implemented. Adaptive water management aims to increase the adaptive capacity of the water system by putting in place both learning processes and the conditions needed for learning processes to take place. As pointed out by Bormann et al. (1993), “Adaptive management is learning to manage by managing to learn.” In this case, learning encompasses a wide range of processes that span the ecological, economic, and socio-political domains in the testing of hard and soft approaches (Pahl-Wostl, 2002; Gleick, 2003). In this respect, adaptive management emphasizes the importance of the management process rather than focusing on goals, but without claiming that the process is an end in itself. It explicitly recognizes that management strategies and even goals may have to be adapted during the process as new information becomes available, and that the quality of the process, e.g., who is involved and which kind of information is taken into account, is essential for the outcomes finally achieved [Pahl-Wostl et al., 2007].
To take into account the different kinds of uncertainties and to implement and sustain the capacity for change, the whole process of policy development and implementation requires a number of steps that are part of an iterative cycle as represented in figure 2.1, all of these steps should be participatory. In the definition of the problem (0), different perspectives need to be taken into account. The design of policies (1) should include scenario analysis to identify key uncertainties and find strategies that perform well under different possible, but initially uncertain, future developments; this is preferable to searching for the best strategy for very specific conditions, e.g., climate, because that strategy may not perform well if those conditions are not met. Policies must be understood as semiopen experiments that require a careful evaluation of potential positive or negative feedback mechanisms by planning and implementing other related policies (1, 2).
Decisions should be evaluated in part by how much it would cost to reverse them. Large-scale infrastructure or rigid regulatory frameworks increase the costs of change, but costs may also be related to a loss of trust and credibility if uncertainties and the possible need for changes are not addressed by the competent authority during policy development (3). The design of monitoring programs should include processes that can pinpoint undesirable developments at an early stage. This might imply different kinds of knowledge, including community-based monitoring systems (3). The policy cycle must include support for institutional settings in which actors assess the performance of management strategies and implement change if needed (4). Continuous replanning and reprogramming based on the results of monitoring and evaluation should be institutionalized (4).

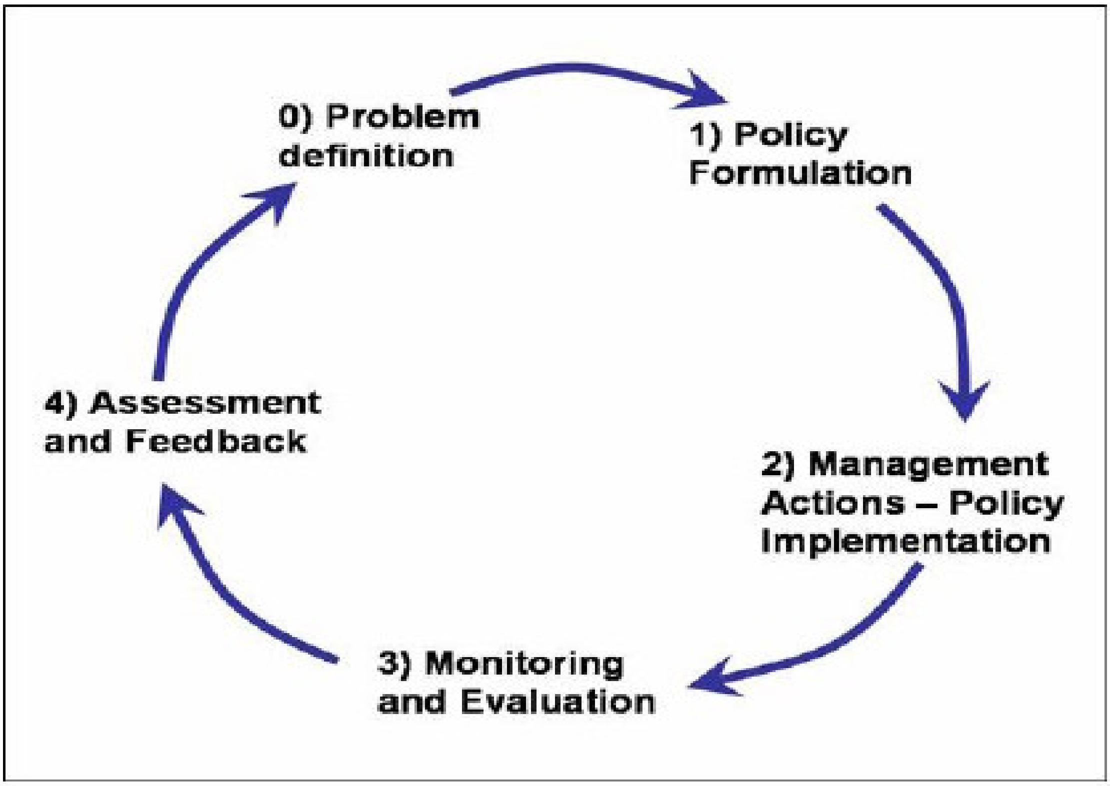


Figure 2.1. Iterative cycle of policy development and implementation in adaptive management [Source: Pahl-Wostl et al., 2007].

2.2 ANALISIS OS WATER RESOURCES SYSTEMS

Water resources systems in general may be represented as a circle combining components interacting with the environment; where the inputs can be classified into three types of variables; controlled, partially controlled and uncontrolled, while the resulting outputs are categorized; desirable, undesirable and neutral. The real challenge is how to convert undesirable and neutral outputs to desirable [Hall and Dracup, 1970]. This may be obtained by controlling the partially controlled and if possible partially control the uncontrolled inputs. There must be feedback between inputs and outputs and vice-versa. The schematic of figure 2.2 illustrates a simple representation to a system subject to input variables and the interactive system components produce a set of outputs [Hall and Dracup, 1970].
The process of assigning certain values by decision makers to the controlled and/or partially controlled input variables is termed the water policy space.
Accordingly, systems modeling include three basic steps:

a- Systematic methods of analyzing systems of independent components to identify and evaluate alternative designs and operating policies.
b- A framework for assisting those responsible for making decisions, solving problems, or gaining an improved understanding.

c- A process intended to focus and force clear thinking about "Large, Complex" systems problems and to promote more informed decisions.

Therefore, the systems analysis exercises follows a certain procedure in order to achieve its practical targets:

a- Problem recognition, definition and bounding.
b- Identification of goals and objectives.
c- Generation of alternatives and evaluation.
d- Decision, implementation and monitoring.

It should be emphasized that re-evaluation of any of the steps may be necessary, if the process may reveal any non-realistic results or ambiguities.

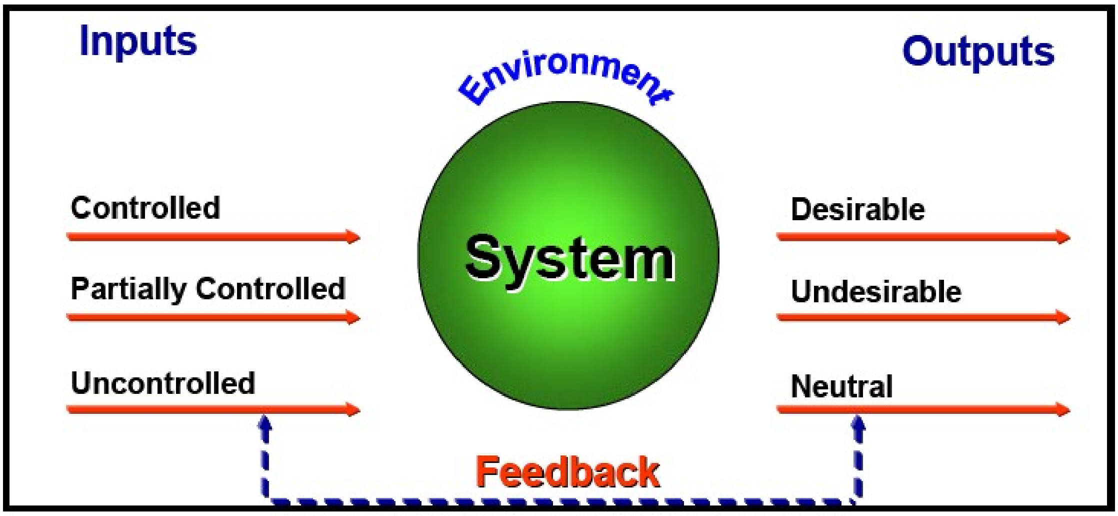


Figure 2.2. All possible feasible is termed the policy space and physical space [Hall and Dracup, 1970].

2.3 MODELING WATER RESOURCES SYSTEMS

It is well known that modeling is only part of the entire planning and management process. The role of models accordingly are:

a- Generate information, and predict impacts.
b- Help identify and evaluate alternatives to increase understanding.
c- Identify trade-offs among goals, objectives, interests, and data needs.

It is possible to increase the use and usefulness of water resources models through interactive programming. Practical experience indicates that policy makers have to know basic principles about models and modeling which include, but not limited to:

a- When modeling might help them to make more informed decisions.
b- Be aware of type of analysis, simple model development and analysis.
c- Maintain considerable but informed scepticism.
d- Realize that models provide only information [El-Kady and El-Shibiny, 2004].

2.3.1 Types of Simulation Models

Simulation models can be statistical or process oriented, or a mixture of both. Pure statistical models are based solely on data (field measurements). Pure process oriented models are based on knowledge of the fundamental processes that are taking place. The example simulation model just discussed is a process oriented model. It incorporated and simulated the physical processes taking place in the system. Many simulation models combine features of both of these extremes. The range of various simulation modeling approaches applied to water resources systems is illustrated in figure 2.3.

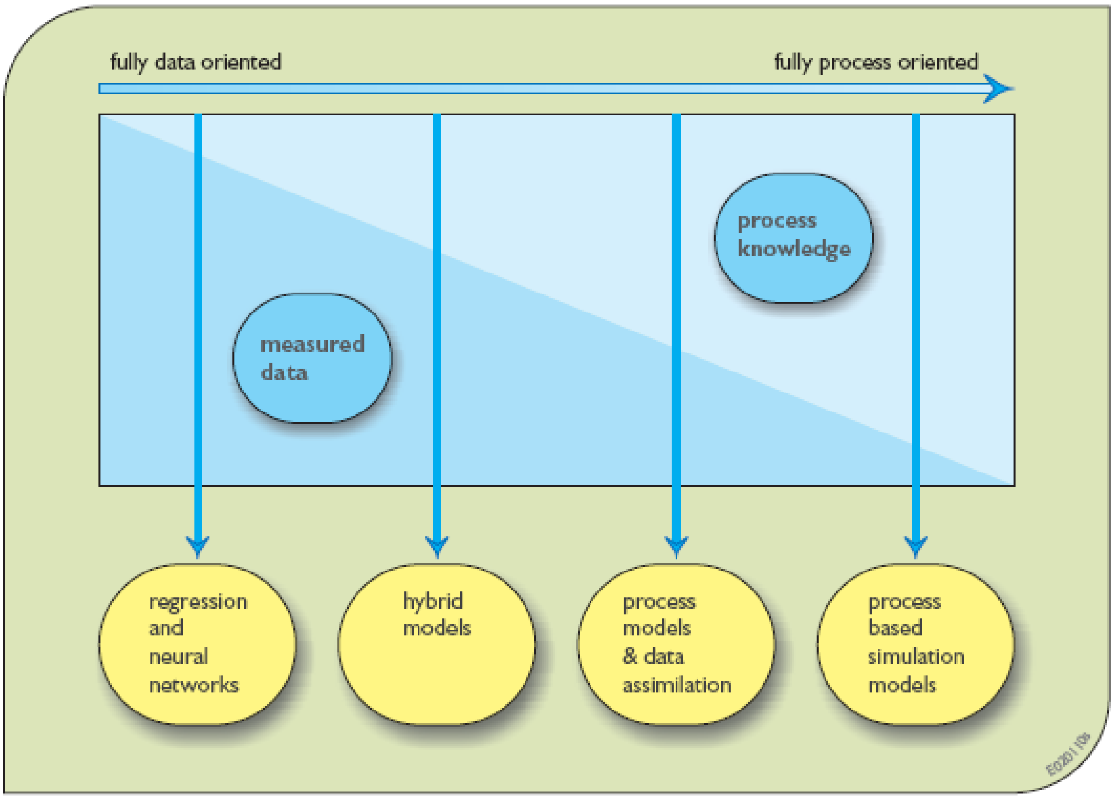


Figure 2.3. Range of simulation models types based on the extent to which measured field data and descriptions of system processes are included in the model [Loucks and Beek, 2005].

Regressions, such as that resulting from a least-squares analysis, and artificial neural networks are examples of purely statistical data driven models. A relationship is derived between input data and output data, based on measured and observed data. The relationship between the input and the output variable values is derived by calibrating a black-box or statistical model with a predefined structure unrelated to the actual natural processes taking place. Once calibrated, the model can be used to estimate the output variable values as long as the input variable values are within the range of those used to calibrate the model. Such models are useful when the data base is consistent and the system described is homogeneous.

Hybrid models incorporate some process relationships into regression models or neural networks. These relationships supplement the knowledge contained in the calibrated parameter values based on measured data.
Most simulation models frequently containing process relationships include parameters whose values need to be estimated. This is called model calibration or optimum parameters estimation. This requires measured field data. These data can be used for initial calibration and verification, and in the case of ongoing simulations, for continual calibration and uncertainty reduction. The latter is sometimes referred to as data assimilation.
Simulation models of water resources systems generally have both spatial and temporal dimensions. These dimensions may be influenced by the numerical methods used, if any, in the simulation, but otherwise they are usually set, within the limits desired by the user. Spatial resolutions can range from 0 to 3 dimensions. Models are sometimes referred to as quasi 2- or 3-dimensional models. These are 1 or 2-dimensional models set up in a way that approximates what takes place in 2- or 3-dimensional space, respectively. A quasi-3D system of a reservoir could consist of a series of coupled 2D horizontal layers, for example.
Simulation models can be used to study what might occur during a given time period, say a year, sometime in the future, or what might occur from now to that given time in the future. Models that simulate some particular time in the future, where future conditions such as demands and infrastructure design and operation are fixed, are called stationary or static models. Models that simulate developments over time are called dynamic models.
Static models are those in which the external environment of the system being simulated is not changing. Water demands, soil conditions, economic benefit and cost functions, populations and other factors do not change from one year to the next. Static models provide a snapshot or a picture at a point in time. Multiple years of input data may be simulated, but from the output statistical summaries can be made to identify what the values of all the impact variables could be, together with their probabilities, at that future time period.
Dynamic simulation models are those in which the external environment is also changing over time. Reservoir storage capacities could be decreasing due to sediment load deposition, costs could be increasing due to inflation, wastewater effluent discharges could be changing due to changes in populations and/or wastewater treatment capacities, and so on. Simulation models can also vary in the way they are solved. Some use purely analytical methods while others require numerical ones. Many use both methods, as appropriate [Loucks and Beek, 2005].

2.3.2 Types of Optimization Methods

There are many ways to classify various types of constrained optimization models. Optimization models can be deterministic or probabilistic, or a mixture of both. They can be static or dynamic with respect to time. Many water resources planning and management models are static, but include multiple time periods to obtain a statistical snapshot of various impacts in some planning period. Optimization models can be linear or non-linear [El-Kady and El-Shibiny, 2004]. They can consist of continuous variables or discrete or integer variables, or a combination of both. But whatever type they are, they have in common the fact that they are describing situations where there exist multiple solutions that satisfy all the constraints, and hence, there is the desire to find the best solution, or at least a set of very good solutions.
Optimization models can be based on the particular type of application, such as reservoir sizing and/or operation, water quality management, or irrigation development or operation. Optimiza-

tion models can also be classified into different types depending on the algorithm to be used to solve the model. Constrained optimization algorithms are numerous. Some guarantee to find the best model solution and others can only guarantee locally optimum solutions. Some include algebraic 'mathematical programming' methods and others include deterministic or random trial-and-error search techniques. Mathematical programming techniques include Lagrange multipliers, linear programming, non-linear programming, dynamic programming, quadratic programming, fractional programming and geometric programming [Loucks and Beek, 2005].

2.4 THE CASE OF AHDR

This section is divided to three main parts, the first part is about the Nile basin which consider the main source of water supply to Egypt and the impact of climate change on the Nile inflows, the second part describes the technical and ecological impacts of the AHD, Egypt's water supply and demands in the present and the future are represented in the third part.

2.4.1 The Nile Basin

The Nile basin is one of the greatest basins in the world with a drainage area of about 2.9 million km^2, river length of 6700 Km, mean annual discharge at Aswan of 84 BCM and mean annual sediment load of 124 million tons/year. The Nile basin extends from 4^o S to 31^o N latitude, and from 21^o 30' to 40^o 30' E longitude [Strzepek et al., 1996]. The lakes in the Nile basin have a total area of 81,550 Km^2 and its swampy reaches amount to 67,000 Km^2. The Nile basin covers parts of ten African countries as illustrated in figure 2.4 (Burundi, Egypt, Eriterea, Ethiopia, Kenya, Rawanda, Sudan, Tanzania, Uganda and Democratic Republic of Congo) [NBCBN, 2005]. The Nile River has two main tributaries, the White Nile which originates from the Lake Victoria basin, and the Blue Nile which has its sources on the Ethiopian Plateau. The two rivers join at Khartoum, the capital of Sudan. The Nile river basin includes a wide range of climatological conditions and land-use, from tropical rainforest in the Lake Victoria area, the wetlands in southern Sudan, pastoral plains and rough mountains in Ethiopia till the extreme aridity of northern Sudan and Egypt.

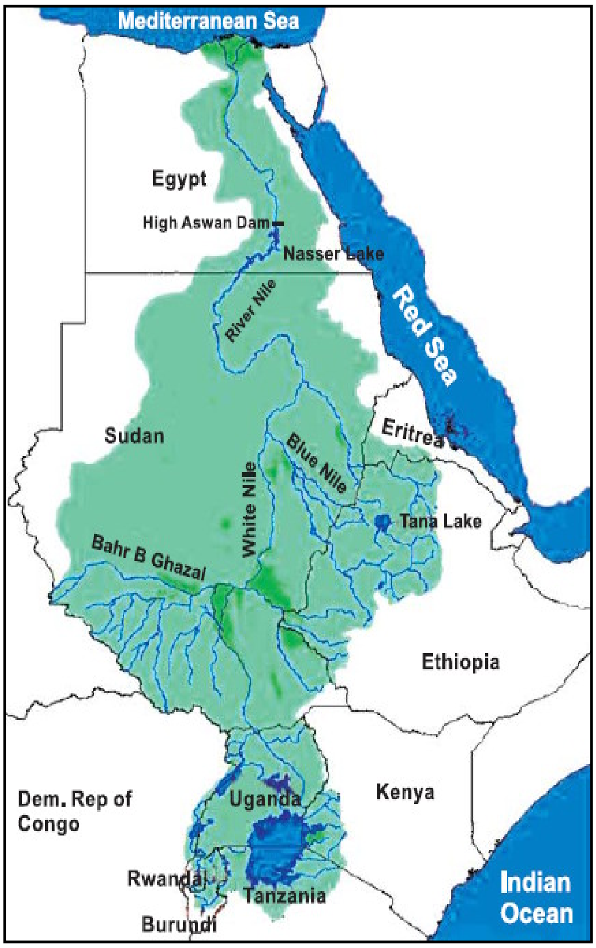


Figure 2.4. The Nile Basin [Source: ICID, 2005].

Compared to many other major rivers in the world the Nile has not undergone major 'developments' yet, except the lower reach in Egypt which has been brought under nearly full control by the construction of the AHD [MWRI, 2005].

2.4.1.1 Hydrology of the river

The River Nile is considered to be the longest river in the world and has one of the largest catchment areas; yet in terms of flow it is exceeded by many rivers. The Amazon has an annual flow of 3000 BCM, the Congo 1250 BCM, the Niger at its mouth 218 BCM as compared with the Nile having an average annual flow of 84 BCM at Aswan [Ibrahim, 1985]. This is due to the fact that the catchment area in the Equatorial lakes region and the Ethiopian plateau, contributing effectively to the run-off, represent about 30% of the total catchment of the Basin. Moreover, the passage of the upper White Nile through lakes and swamps and the flow of the main Nile across the great North African desert, contributes considerably to the reduction of the river flow. This situation is very well illustrated when considering the water balance of the Nile Basin. The overall run-off coefficient of the Nile basin as calculated at Aswan is 6%. The catchment area from Khartoum to the Mediterranean hardly contributes any flow to the Nile.
The Nile is also known for its marked seasonal and annual variations. The variation in discharge is illustrated by the fact that more than 80% of its annual flow occurs from August to October as shown in figure 2.5 and only 20% occurs during the remaining nine months. It is also interesting to note that the annual discharge of the Nile for the year 1913-14 was 41 BCM as compared to 151 BCM in 1878-79, while the average annual flow is 84 BCM.

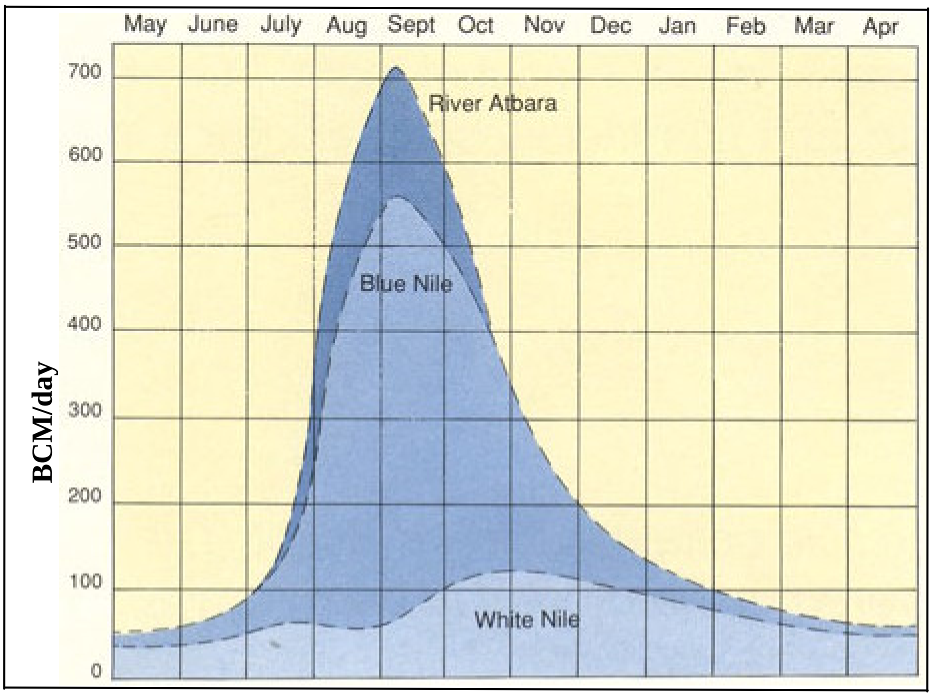


Figure 2.5. The flow rate of the Nile at different times at the year [Source: Arthur, 2004].

The percentage contribution of the main tributaries of the Nile is as follows [Ibrahim, 1985]:

Blue Nile	59%
Sobat	14%
River Atbara	13%
Bahr El Jebel	14%

In other words 85% of the flow of the Nile comes from the Ethiopian plateau and only 15% comes from the other African riparian countries.
During flood time the percentage contribution of the tributaries is as follows:

Blue Nile	68%
River Atbara	22%
Sobat	5%
Bahr El Jebel	5%

In other words, during flood 95% of the water comes from the Ethiopian highlands and only 5% comes from East Africa. During the low flow period 60% of the water comes from Ethiopia and 40% from East Africa. The low contribution of the White Nile to the Main Nile is attributed to the great amount of water which is lost by evaporation in the swamps while the Ethiopian plateau acts efficiently for draining the water to the Nile.

2.4.1.2 Regulation rules for the reservoirs along the River Nile

The River Nile includes some of single reservoir regulation rules and regional coordination rules. The following section describes some of these rules [Yao and Georgakakos, 2003].

2.4.1.2.1 Reservoir release-elevation rule

This is a single reservoir regulation rule. The release of a reservoir at any particular time period T (month, 10 days, day) is determined by its elevation:

$$u_i (T) = g (h_i (T))$$

Where u is the discharge in cubic meters per second, h is the reservoir elevation in meters, and g is a function provided by the user. Currently, this type of rule is used by the Equatorial Lakes. figures 2.6 through 2.8 show these curves.

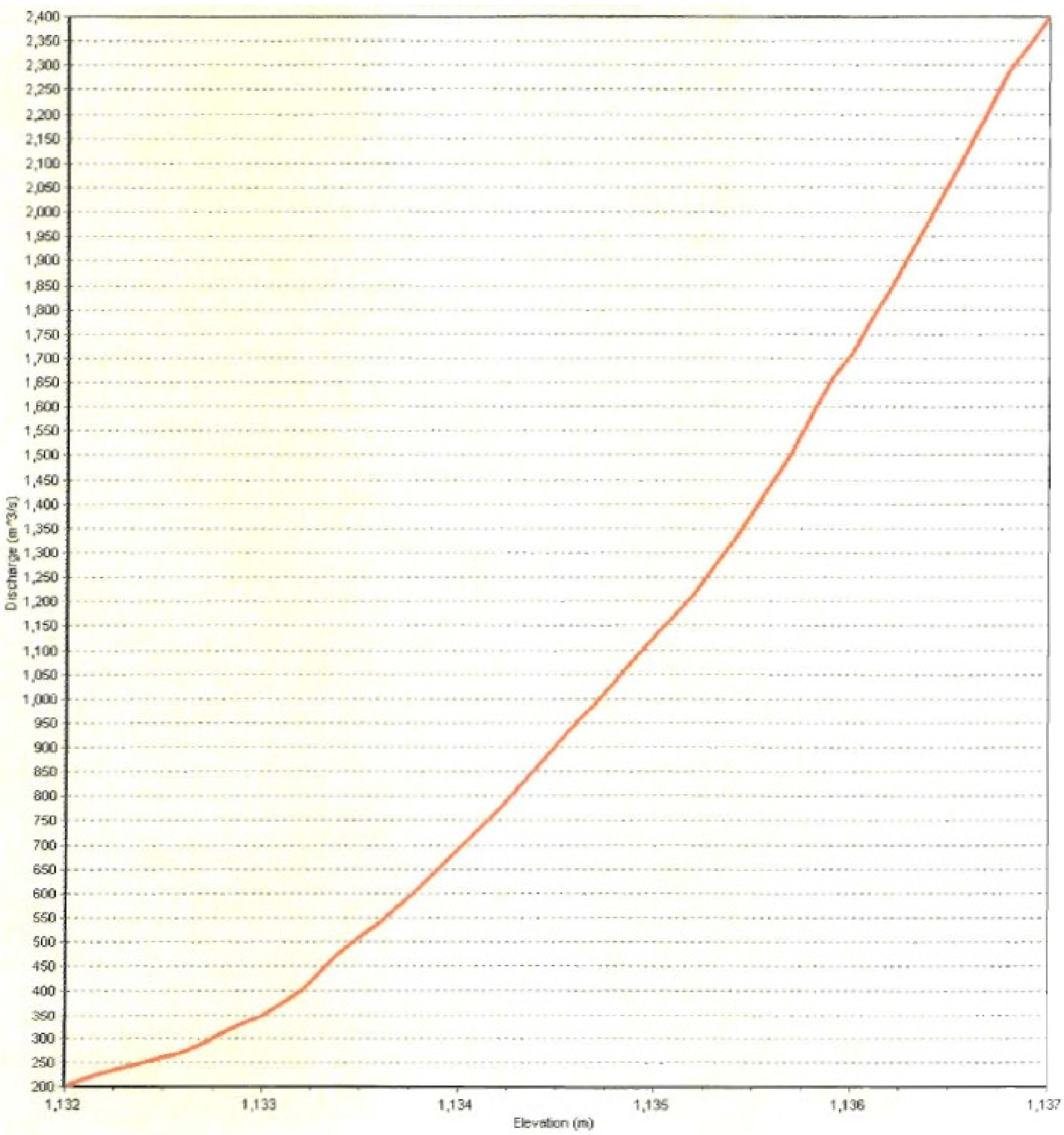


Figure 2.6. Natural outflow curve for Lake Victoria [Source: Yao and Georgakakos, 2003].

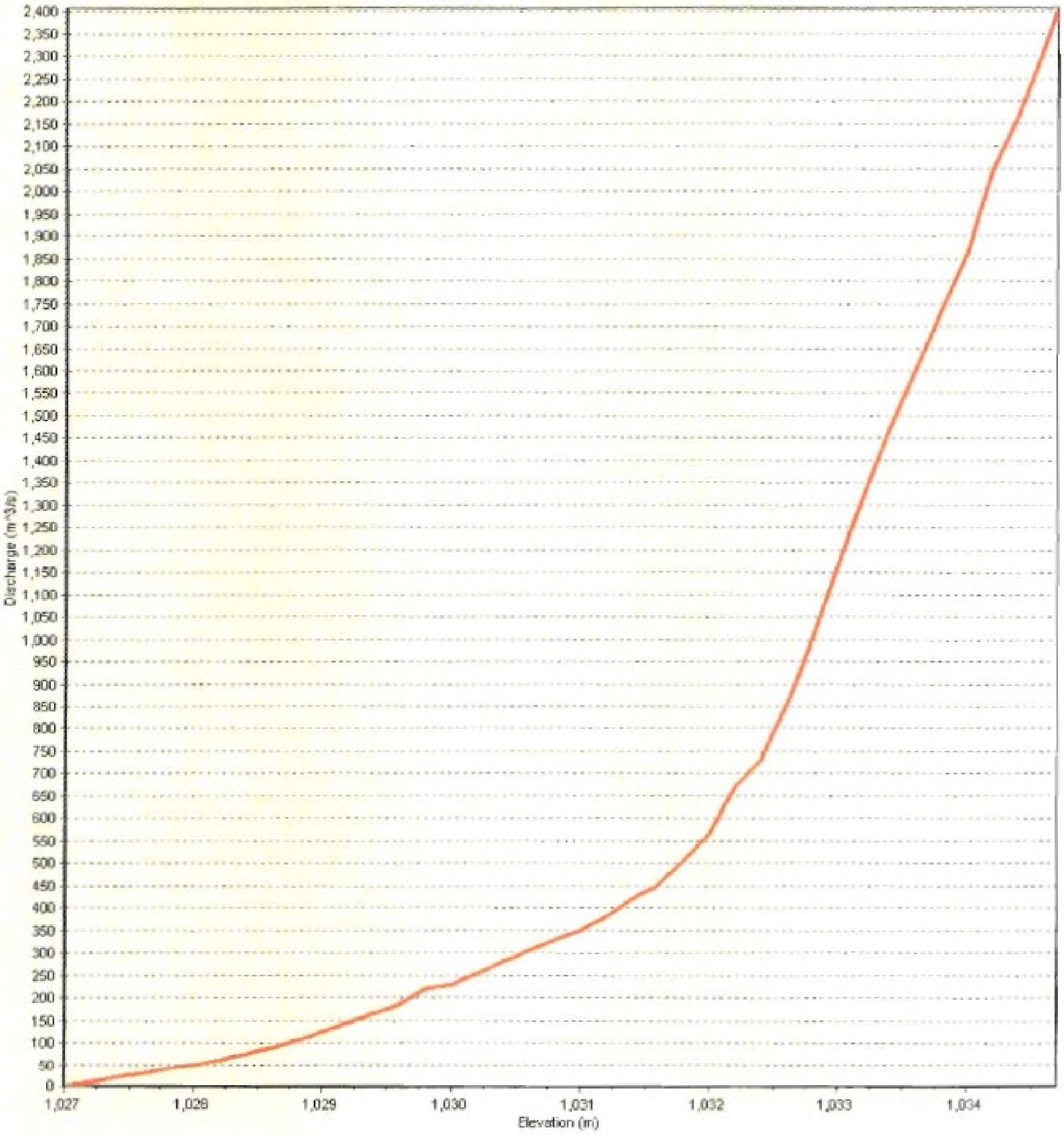


Figure 2.7. Natural outflow curve for Lake Kyoga [Source: Yao and Georgakakos, 2003].

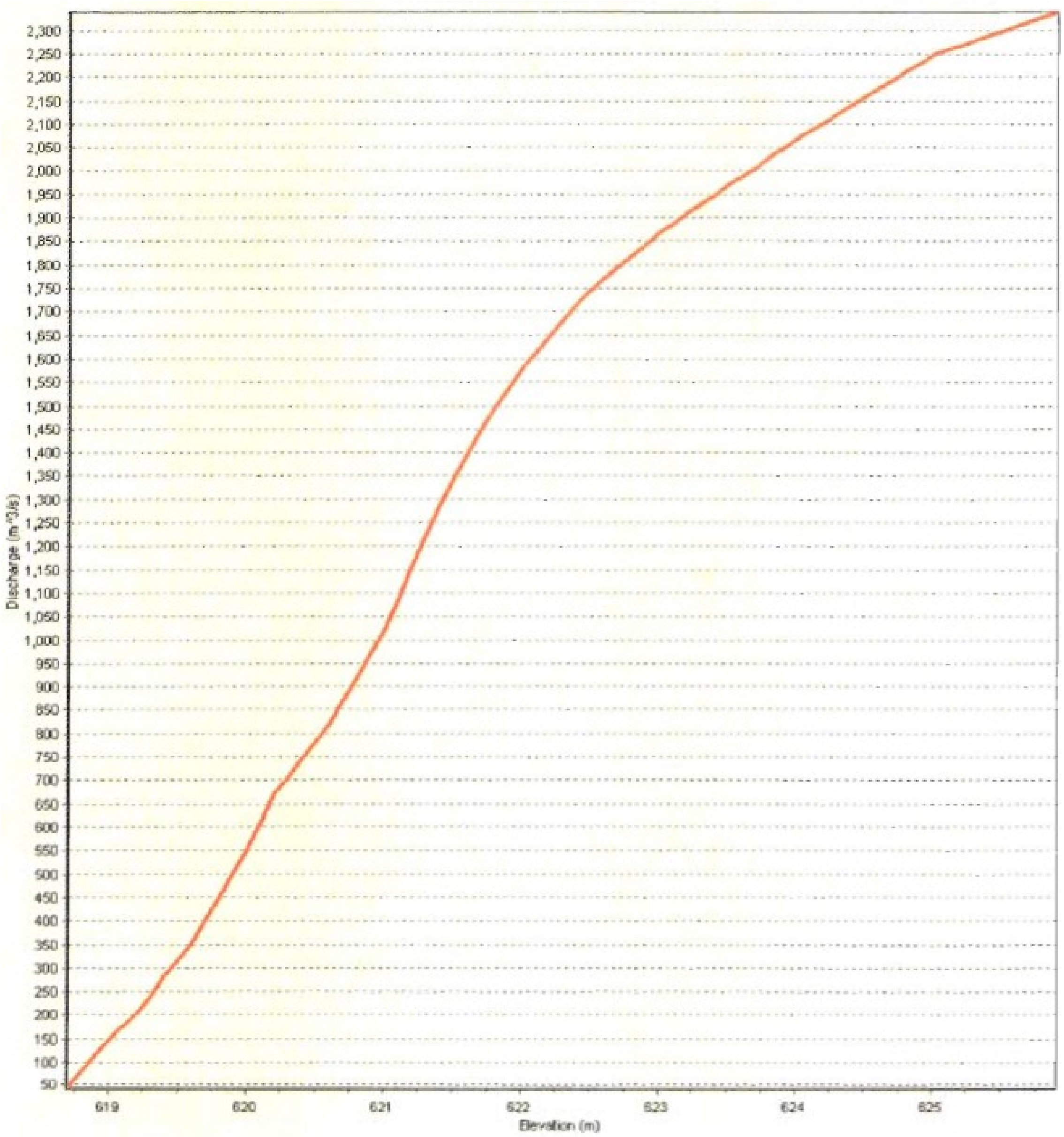


Figure 2.8. Natural outflow curve for Lake Albert [Source: Yao and Georgakakos, 2003].

2.4.1.2.2 Target reservoir elevation rule

This rule also applies to single reservoirs. The rule tries to follow target reservoir elevation sequence over time. The release for each period is determined by:

$$u_i\,(T) = S_i\,(T) + W_i(T) - e_iA_i\,(T) - D_i(T) - S_i\,(H_{tgt}\,(T+1))$$

Where S is the storage, $H_{tgt}(T+l)$ is the target elevation at the end of the period, W is the inflow, D is the is the withdrawal, and eA is the evaporation loss (as a product of net evaporation rate, surface area, A). A typical 10-day target elevation sequence for Gebel El Aulia is shown in figure 2.9.

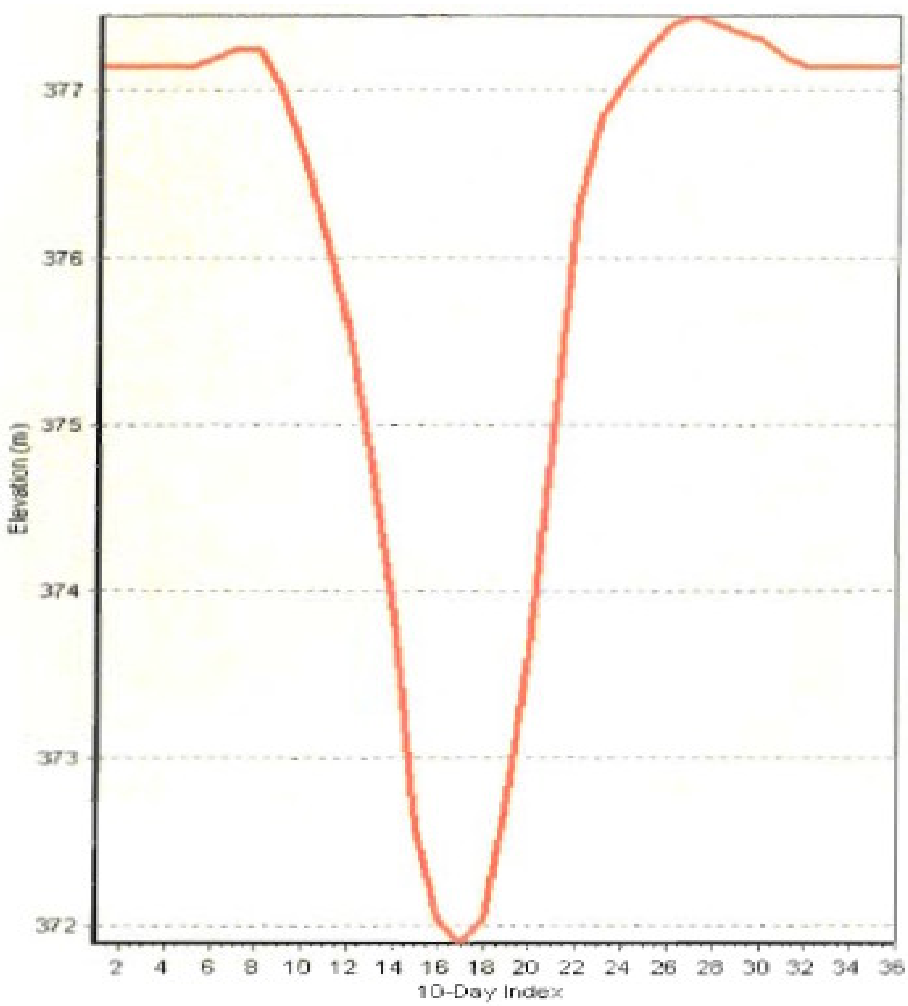


Figure 2.9. Gebel Al Aulia Target Elevation [Source: Yao and Georgakakos, 2003].

2.4.1.2.3 Target release rule

This single reservoir regulation rule operates the reservoir to follow a target release sequence. The release is simply equal to its target value:

$$u_i\,(K) = R_{itgt}\,(K)$$

Where R_{itgt} (K) is the target release for period K. The normal operation of the AHD follows this type of rule, the target values being the 10-day downstream irrigation demands. A sample target release sequence is shown in figure 2.10.

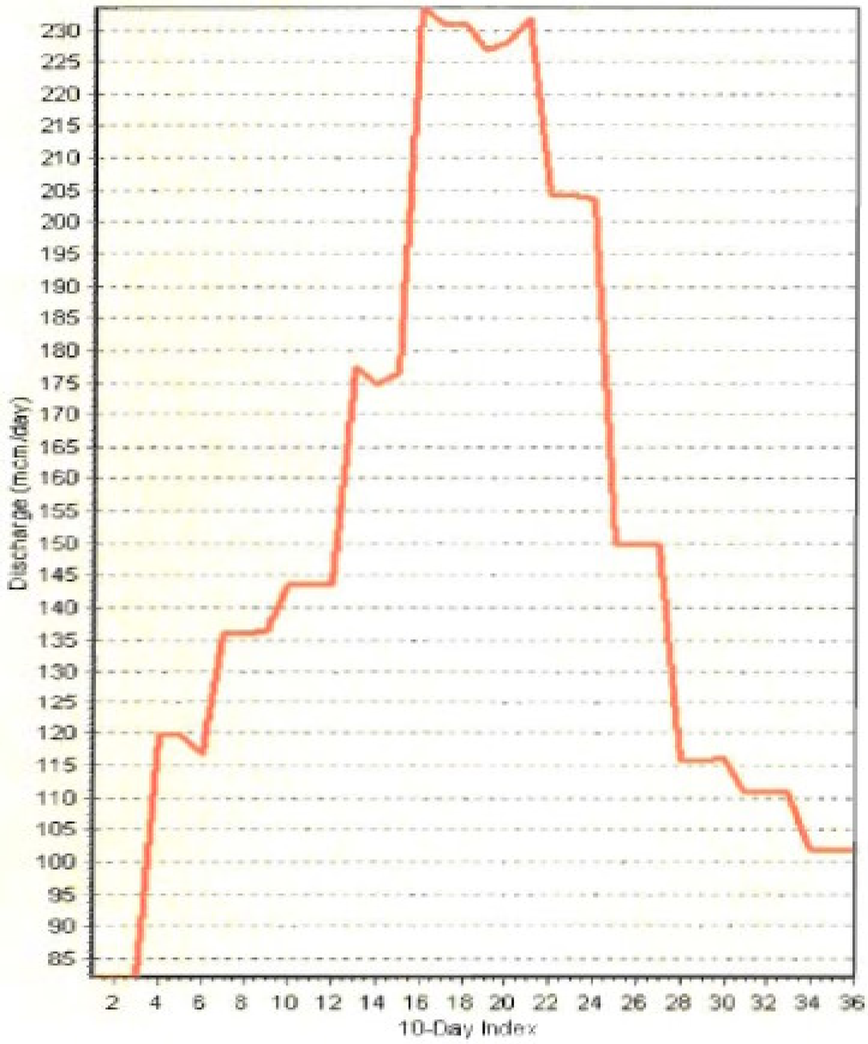


Figure 2.10. Sample AHD 10-day Demands [Source: Yao and Georgakakos, 2003].

2.4.1.3 Climate change in Nile Basin

The Nile's hydrologic characteristics are highly sensitive to climate change. The Nile is marked by 2 topographic extremes: mountainous plateaus and flat plains. The Equatorial Plateau and its system of lakes have a very delicate water balance, with direct evaporation from the lake surfaces almost equal to the direct precipitation onto the lakes. Although the net water gain per unit area is small, the area of the lakes is large, so the direct lake water supply plus the tributary inflow results in a large volumes of water. However, a small shift in either rainfall or evaporation can result in significant changes in Lake Victoria, as observed in the 1960s when a historically rapid rise and increase of lake discharge occurred. Piper et al. (1986) observed that the 1961-1964 rises are not unique and that similar fluctuations have occurred in the past. Indeed, there is some evidence from paleo climatic records that in recent times the Victoria Basin became actually closed with no outflow.

2.4.1.3.1 Rainfall variability in the headwaters of the Nile

Climate characteristics and vegetation cover in the Nile Basin are closely correlated with the amount of precipitation. Precipitation is to a large extent governed by the movement of the Inter-Tropical Convergence Zone (ITCZ) and the land topography. In general precipitation increases southward and with altitude (note the curvature of the rainfall isoheights parallel to the Ethiopian Plateau). Precipitation is virtually zero in the Sahara desert, and increases southward to about 1200–1600 mm/year on the Ethiopian and Equatorial lakes Plateaus. Two oceanic sources supply the atmospheric moisture over the Nile basin; the Atlantic and the Indian Oceans, respectively.
The seasonal pattern of rainfall in the basin follows the movement of the ITCZ. The ITCZ is formed where the dry northeast winds meet the wet southwest winds. As these winds converge, moist air is forced upward, causing water vapor to condense. The ITCZ moves seasonally, drawn toward the area of most intense solar heating or warmest surface temperatures. Normally by late August/early September it reaches its most northerly position up to 20^o N. Moist air from both the equatorial Atlantic and the Indian Ocean flows inland and encounters topographic barriers over the Ethiopian Plateau that lead to intense precipitation, responsible for the strongly seasonal discharge pattern of the Blue Nile. The retreat of the rainy season in the central part of the basin from October onwards is characterized by a southward shift of the ITCZ (following the migration of the overhead sun), and the disappearance of the tropical easterly jet in the upper troposphere [Mohamed et al., 2005].
Figure 2.11 shows annual rainfall averaged over the Blue and White Nile, respectively (represented by rain gauges located in or close to the Blue Nile and Lake Victoria, respectively) from 1905 to 1999. In the Blue Nile basin a slightly increasing trend occurred between 1905 and 1965 followed by a prolonged decline which bottomed out in 1984 and recovered during the 1990s with 1996 the wettest year since 1964 (33 years). In contrast, rainfall over Lake Victoria shows a moderate increasing trend up to 1960 followed by a prolonged increase in annual rainfall due to a combination of extremely wet years, e.g. 1961, 1963 and 1977 and small increases in other years. Annual rainfall over much of the Lake Victoria region increased from 1931-60 to 1961-90 by roughly 8% [Conway, 2005].

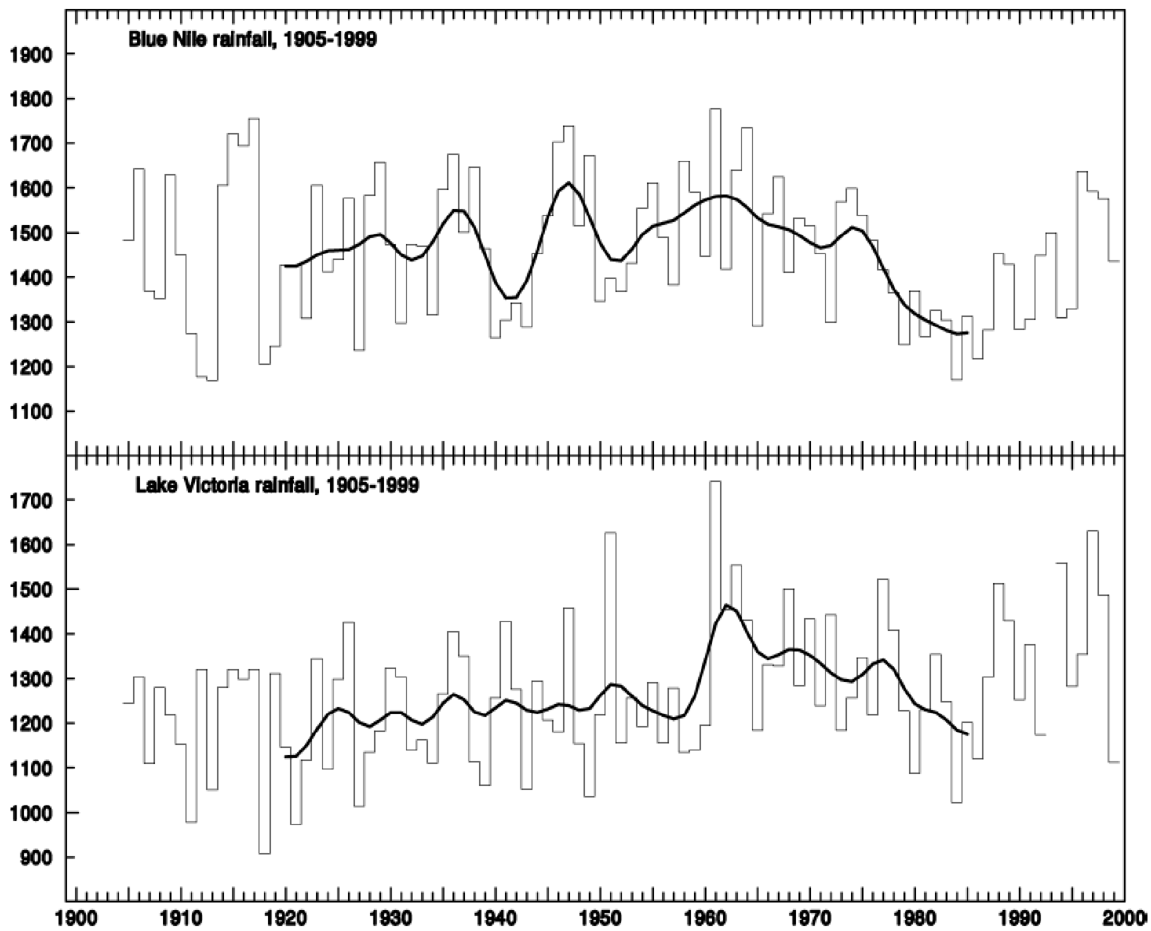


Figure 2.11. Average annual rainfall 1901-99 in the Blue Nile and Lake Victoria catchments [Source: Conway, 2005].

2.4.1.3.2 Variability in White and Blue Nile flows – impacts on Nile river flows

Figure 2.12 shows annual Blue Nile flows, Lake Victoria levels and Nile flows at Aswan. Runoff in the Blue Nile basin amounts to 45.9 km^3 (equivalent to 1456 m^3s^{-1}), a depth of 261 mm (1961-1990), and a runoff coefficient of 18%. Between 1900 and 1997 annual river flow has ranged from 20.6 km^3 (1913) to 79.0 km^3 (1909), and the lowest decade-mean flow was 37.9 km^3 from 1978-87. A significant and sustained increase in Lake Victoria levels and outflows occurred in late 1961.

Lake Victoria levels increased by 2.25 m from 1961 to their peak in 1964 equivalent to an increase in storage volume of 151 BCM and decreased steadily except for short-lived rises in 1978-79, 1990-91 and 1997-98 and they remain well above their pre-1961 levels. Lake Victoria outflows roughly doubled from 1931-60 to 1961-90 [Conway, 2005]. Downstream the long record of Nile flows into Egypt integrates the effects of the Blue Nile and Lake Victoria along with other

lakes and wetlands on the White Nile system and varying contributions from other tributaries. Nevertheless, because of the large proportional contributions to the Nile from the Blue Nile and Lake Victoria their variability is strongly reflected in the Nile flow records. The low Blue Nile flows during the 1980s were partially offset by higher White Nile flows since the 1960s but their effects were still apparent in the Nile flows which reached their second lowest point in 1984.
The recovery of rainfall in the Blue Nile during the 1990s is also reflected in the Nile flows. The period of high flows prior to 1899 has been the subject of a number of studies especially concerning the accuracy of the early gauge data but the change is real [Conway, 2005]. The reduction in flows after 1899 represents a marked change in the regime and the monthly flow patterns and anecdotal evidence suggest that the high flows were due to contemporaneous high flows in the Blue and White Niles.

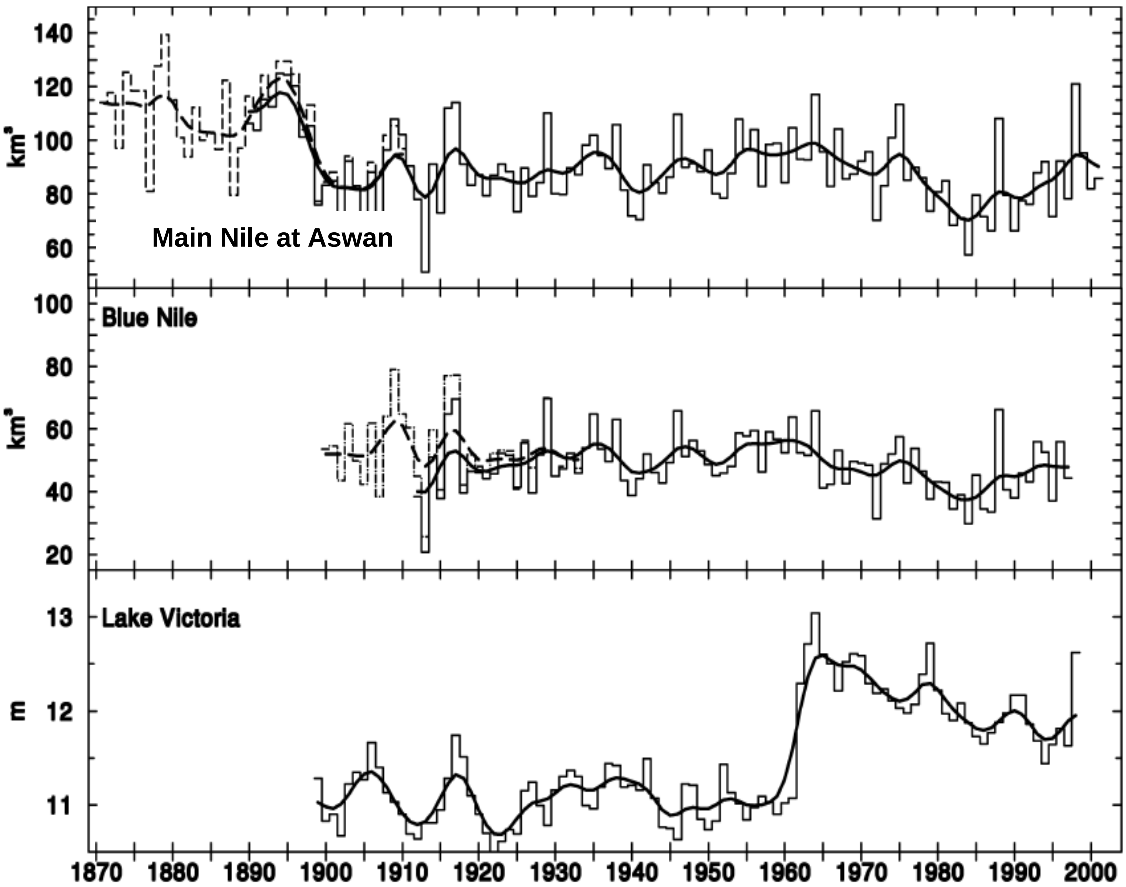


Figure 2.12. Average river flows in the Main and Blue Nile and lake levels in Lake Victoria [Source: Conway, 2005].

2.4.1.3.3 The significance of warming trends for increasing evaporative losses in the Nile basin

There remains low confidence in the direction and magnitude of future rainfall change in the basin, however, the observed regional warming and the high confidence (in IPCC terms) that this will continue at an increasing rate makes it prudent to review the possible effects of higher temperatures on surface water resources in the basin. That there are large expanses of open water (Lake Victoria alone is roughly 67000 km^2) and wetlands, along with reservoirs. eg. evaporation from the AHD is over 10% of the Nile flow.

Losses to open water evaporation in Lake Victoria and the Ugandan Nile lakes, although lower than rainfall amounts are huge in volumetric terms. However, it is reasonable to speculate that some of this moisture may be recycled in the form of rainfall in the region. Evaporation rates begin to exceed rainfall when both the White and Blue Niles enter Sudan. Roughly half of the inflows to the Sudd wetland system in Southern Sudan are lost to evaporation and transpiration (annual Penman evaporation is 2150 mm per year). Evaporation from the Blue Nile river between Roseires and Khartoum is roughly 2 BCM (624 km length, 300 m width), Roseires and Sennar reservoirs have evaporation losses of roughly 0.5 BCM each. On the White Nile, losses from north of the Sudd to just south of Khartoum (a distance of 840 km) are roughly 2 BCM and the large surface area of the Jebel Aulia reservoir loses roughly 2.5 BCM. Channel losses from Khartoum to Dongola, close to the AHDR, are roughly 2.4 BCM due to evaporation rates of 2700 mm over a channel length of 1500 km with average width of 600 m [OECD, 2004]. Finally, in the AHDR and throughout Egypt, evaporation plays a critical role in water resources management. For the AHD alone the estimated evaporation is around 10 BCM (2700 mm evaporation).

2.4.1.3.4 Using climate change to predict Nile flows

In order to assess how climate change might affect operation strategies of the AHDR, it is then necessary to predict Nile floods for the next one hundred years in order to have a knowledge of the amount of supply to the reservoir in future years. This is done by looking at historical Nile flows and using general circulation models to assess effects of climate change. Several studies have been done to assess the impact of climate change on the River Nile. Table 2.1 summaries some climate change studies on the Nile:

Authors	Study objective	Conclusions
Gleick (1991)	Applied an annual water balance model to three sub basins of the Nile basin, the Upper White Nile, Sobat and Blue Nile/Atbara.	The model produced a 50% reduction in runoff in the Blue Nile catchment due to a 20% decrease in rainfall.
Hulme (1994)	Reviewed of future changes in temperature and rainfall based on GCM results for the Nile basin.	Qualitative discussion of their implications for Nile flows.
Onyeji and Fischer (1994)	Economic analysis of potential impacts of climate change in Egypt. Their analysis did not incorporate climatically induced changes in Nile supply.	Projections indicated a decline in agricultural and nonagricultural self-sufficiency and highlighted a number of potentially negative effects of climate change.
Strzepek (1995)	used three GCMs (UKMO, GFDL and GISS) with doubled global atmospheric concentrations (2 x CO2) to predict Nile flow changes at the AHD for 2060.	In the results, GISS was the wettest model and had a 30% increase in annual streamflow; whereas for UKMO, there was a 12% decrease; and for the driest model GFDL, there was a 78% decrease.

Conway and Hulme (1996)	Used hydrologic models of the Blue Nile and Lake Victoria to assess impacts of future climate change on Nile flows. Sensitivity analysis of hypothetical changes in rainfall and evaporation, and a set of seven equilibrium GCM scenarios for 2025.	They obtained a range (due to differences between GCM scenarios) of –9% to +12% change in mean annual Nile flows for 2025. Results were used to estimate changes in the availability of Nile water in Egypt based on the Nile Waters Agreement.
Strzepek and Yates (1996)	Spatially aggregated monthly water balance model used to explore the sensitivity of Nile flows to climate change.	Divergence between climate model results for the Nile basin; from a sample of four models two produced increases and two produced decreases in flows.
Yates and Strzepek (1998a)	Follow-up study to Strzepek and Yates (1996).	They found declines up to -9% in the annual flow at the AHD by 2060 for doubled CO_2, but they found increases for the GISSA and UKMO models for the same period which produced about 40% increase in annual flow at the AHD.
Yates and Strzepek (1998b)	Reanalyzed the results of their CO_2 doubling scenarios	They found that five of six GCMs showed increased streamflows at the AHD for the 2060s (roughly the time for CO_2 doubling) with increases as much as 137% for GISS. Only one GCM (GFDL) showed a decline in annual flow at Aswan (-15%) relative to the long-term average Nile flow.
Arnell, (1999)	Studied the relationship between climate change and global water resources.	Arnell's study suggested that precipitation in the Nile basin would increase by about 10% by 2050, but he suggested that this increased precipitation would be offset by increased evapotranspiration, implying that the net effect on the main stem flow might be insignificant.
Sene (2000)	Investigated the influence of Lake Victoria on flows in the Upper White Nile using a model that represented the main river channel by a series of interconnected lakes and swamps.	The results indicated extreme sensitivity of White Nile flows to changes in Lake Victoria levels and outflows, in particular to variations in direct rainfall on the lake surface.
Strzepek et al. (2001)	Used a sample population of climate change scenarios for the basin that incorporate uncertainties due to differences between climate models, climate sensitivity estimates, and emission pathways. They selected nine representative scenarios from the full range to produce Nile	A propensity for lower Nile flows (in 8 out of 9 scenarios). The wet scenario only produced moderate increases from the 2040s onwards, whilst 3 (4) of the flow scenarios produce large and rapid changes in flows of the order of 40–50% (20–40%) reductions in flow by 2025 (2020) and over 60% (roughly 30–60%)

	flow scenarios using a suite of water balance models.	by 2050.
Yohe et al. (2003)	Developed the approach in Strzepek et al. (2001). Vulnerability to climate variability and change is considered as a function of exposure, sensitivity and adaptive capacity. They select two critical assumptions in their economic scenarios; that Egyptian policy will maintain food sufficiency and macroeconomic vitality as prime objectives. They define a set of socioeconomic scenarios used to drive an economic model for Egypt.	The results focus on 2067, a point when all the climate scenarios are fairly divergent, and use food self sufficiency and the sum of food and consumable consumption to reflect critical components of domestic welfare. Results are presented with and without Nile flow scenarios and with and without adaptations based around micro solutions to adopt drip irrigation and recycle municipal water, and macro-level solutions represented by large groundwater drilling projects in the Sahara.
Conway (2005)	Used the Nile basin as a unit of analysis to explore processes of climate variability, climate change and adaptation acting across a range of spatial and temporal scales. The aim was to use empirical observation to inform practical approaches to adaptation in the basin. The analysis considered variability during the recent past, beginning around the 1860s, when regular measurement of Nile flows began at Aswan.	The effects of climate variability, principally rainfall variability in the Ethiopian highlands and Lake Victoria, are shown to have caused significant interannual and interdecadal variability in Nile flows with major implications for water resources in Egypt.
Beyene et al. (2009)	Used 11 General Circulation Models (GCMs) and two global emissions scenarios (A2 and B1) archived for the 2007 IPCC report to assess the potential impacts of climate change on the hydrology and water resources of the Nile River basin.	The results showed that, averaged across the multimodel ensembles, the entire Nile basin will experience increases in precipitation early in the century (period I, 2010-2039), followed by decreases later in the century (periods II, 2040-2069 and III, 2070-2099) with the exception of the easternmost Ethiopian highlands which is expected to experience increases in summer precipitation by 2080-2100.

Table 2.1. Previous work on climate change and its impacts on Nile flows.

2.4.2 The Aswan High Dam (AHD)

To improve management of the main water resource in Egypt, the Nile River, it was decided in 1956 by the Egyptian government to build one of the highest dam in the world with one of the biggest man made reservoirs in the world.

The AHD is a rock fill dam retaining the Nile flows at a distance of 7 km south of Aswan. The length of the dam crest is 3600 m and its height is 111 m above the river bed (figures 2.13) [Abu-Zeid and El-Shibini, 1997].

The AHD is equipped with a diversion canal, the diversion canal on the eastern bank of the Nile is composed of an upstream and a downstream canal linked by the main tunnels, which were dug in the rocks underneath the right wing of the dam. The total length of the diversion canal is 1950 m, of which 1150 m are located in the upstream side, 485 m are in the downstream side, and 315 m within tunnels and the hydro-electric power station [Hassan, 1999].

Six spillway tunnels (see figure 2.14) have been constructed to link the upstream and downstream canals. The average length of each tunnel is 282 m with a 15 m circular cross-section of internal diameter lined with reinforced concrete of a minimum thickness of one meter [Hassan, 1999].

Each tunnel is divided vertically into two branches before its connection with the hydroelectric power station. These branches are divided again by a horizontal wall into two water passages, one of them supplies water to power generating units and the other is controlled by sector gates for passing the surplus water needed during the period of peak water requirements. At maximum, the six tunnels were designed to release discharge of 11,000 m^3/s, about 1 BCM/day [Mhmod et al., 2006].

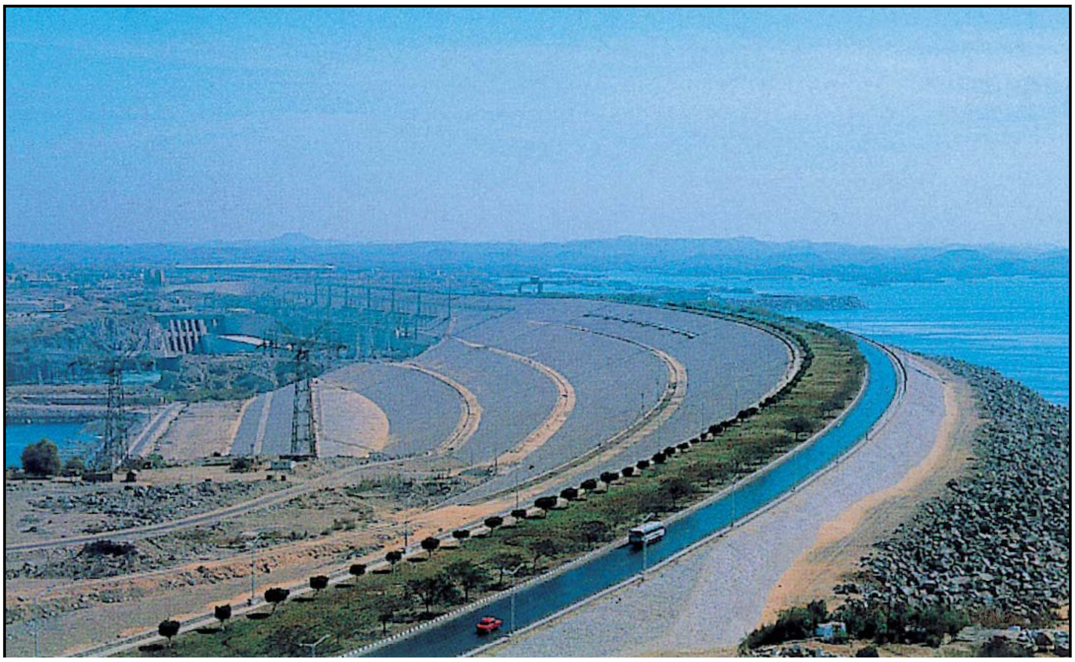


Figure 2.13. The AHD on the Nile River [Source: ICID, 2005].

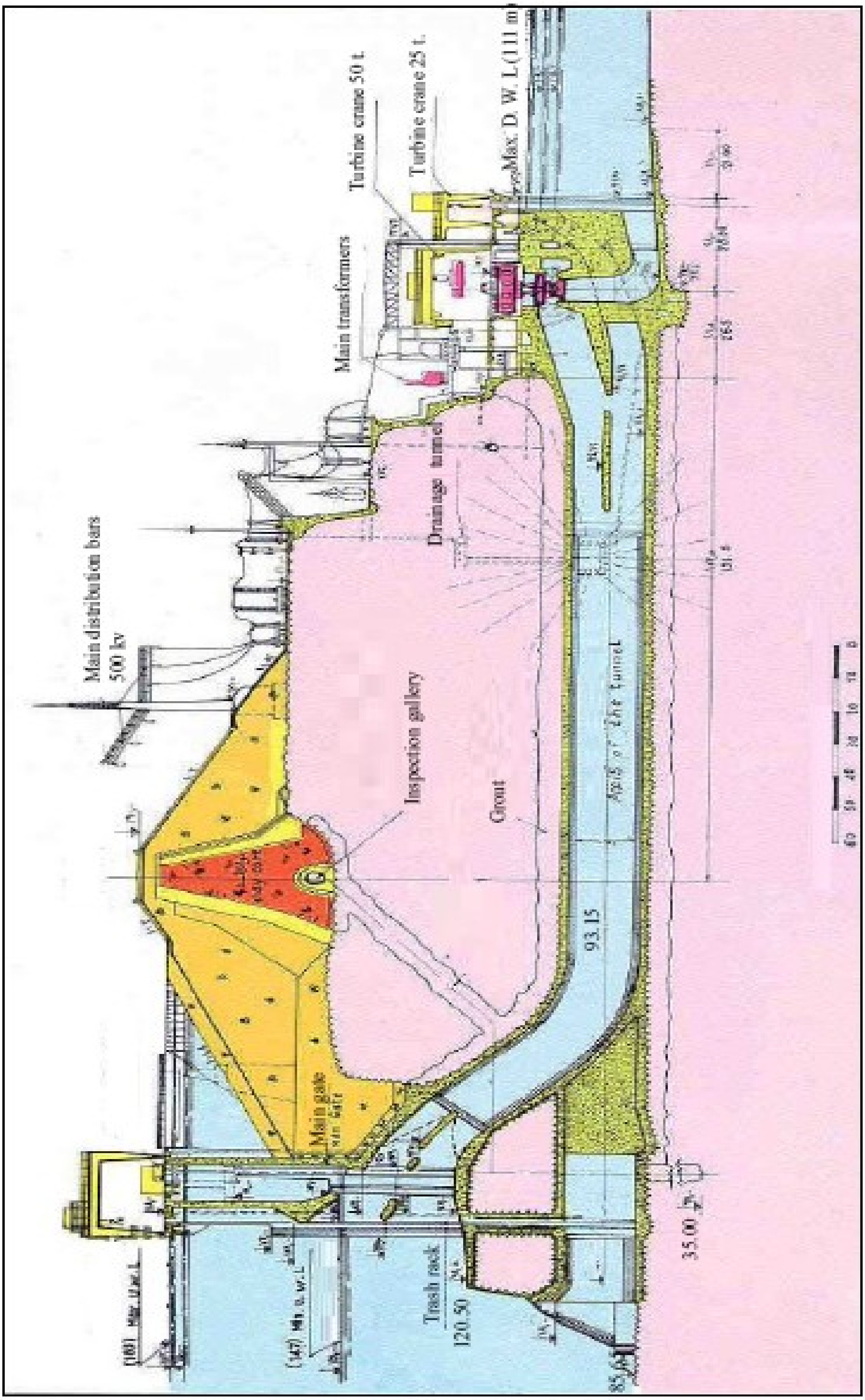


Figure 2.14. Cross section of the AHD [Source: Saad, 2008].

2.4.3 The Aswan High Dam Reservoir (AHDR)

The AHDR is located on the border between Egypt and Sudan between latitudes 21.8 to 24.0°N and longitudes 31.3 to 33.1°E (see figure 2.15). The reservoir is about 500 km long (more than 350 km in Egypt and the rest in Sudan). The reservoir has a volume of 162 BCM and a surface area of about 6500 km^2 at the maximum water level 182 m upstream the dam [MWRI, 2005]. The reliable water supply from the AHDR is estimated as 55.5 BCM/year, based on the average natural flow of 84 BCM/year, reservoir evaporation losses of 10 BCM/year and an allocation of 18.5 BCM/year for the Sudan [MWRI, 2005].

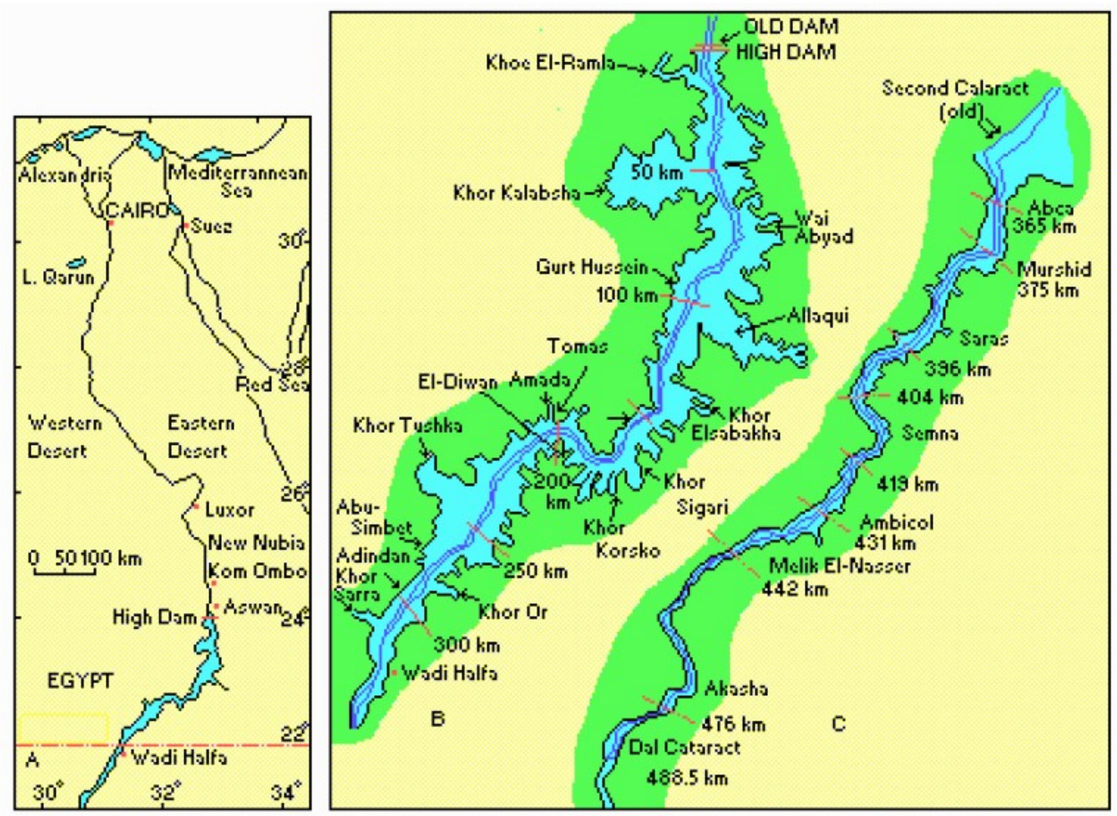


Figure 2.15. Location and extent of the Aswan High Dam Reservoir [*MWRI, 2005].*

2.4.3.1 Socio-economic impacts of the AHDR

2.4.3.1.1 Water security and availability

The long-term or continuous storage policy of the AHDR secured for Egypt an annual water quota of 55.5 BCM instead of a previous mean annual 48 BCM, of which at least 22 BCM were to be released to the Mediterranean during the flood period (August-November). Sudan’s quota was secured at 18.5 BCM instead of 4.5 BCM. This was concluded by a 1959 agreement between

Sudan and Egypt, taking into consideration that the mean annual flow of the Nile at the borders is 84 BCM of which 10 BCM were deducted as mean water losses [MWRI, 2005]. The agreement also contained future plans for development of Nile water by minimizing losses in the Upper Nile catchment and increasing the water yield by 18 BCM annually through three main projects in Bahr El Gabel, Bahr El Gazai and Sobat catchments, respectively.

2.4.3.1.2 The Flood and drought protection

The AHDR is able to store about 2 times the average annual flow volume of the Nile. This over-year storage capacity of the AHDR almost completely stopped flooding of the Nile in Egypt. During the period 1999-2001 some additional releases from the reservoir were needed because of a prolonged high Nile inflow in the previous years and the limited capacity of the Toshka spillway (figure 2.16 shows location of the AHDR and Toshka Lakes). In August/September 2001 about 3125 m^3/s (270 MCM/day) was discharged during 5 days, compared to a normal discharge in that period of the year of 2025 m^3/s (175 MCM/day). Only some minor damage occurred due to flooding of agricultural areas around Giza (a town in Egypt on the west bank of the Nile river, some 20 km southwest of capital Cairo). The planned increase of the discharge capacity of the Toshka spillway will decrease the probability of flooding even more. On the other hand it is expected that climate change may increase the probability of flooding again [MWRI, 2005].

Figure 2.16. Location of the AHDR and Toshka lakes.

On the other side, the large capacity of the AHDR has also caused a substantial reduction of the probability of drought conditions, in particular the occurrence of an unexpected drought that can be regarded as a calamity. Even during the dry period in the 1980’s when the Nile inflow was rather low, the AHDR was able to deliver most of the agreed upon 55.5 BCM. The lowest recorded delivery was 52.5 BCM in 1988. Sliding scales (hedging rules) are applied at the AHD to start reducing the release from the AHDR if the water level falls below a critical level. This aims at saving water to avoid a dry period with hardly any flow which would be disastrous for agriculture. Moreover, a situation with reduced flow at Aswan like the one in the year 1988 can be reasonably well predicted, enabling the agricultural sector to adapt to such situation [MWRI, 2005].

2.4.3.1.3 Hydropower production

The AHD Power plant (fig. 2.17) is situated at the outlets of the tunnels, it is considered as the biggest hydro Power plant station for generating power in Africa for a total of 2.1 GW producing 10 GWh annually as its connected to the grid in the period from 1967 till 1970.
Its power generation has been successfully used in the electrification of Egypt's countryside which comprises more than 4500 villages, and the running of many old and new factories and pumping stations for irrigation and drainage. In addition it has encouraged an increasing pace of industrialization and a rise in the standard of living, promoting culture, education and civilization throughout Egypt, particularly in rural areas. The AHD has moreover improved the efficiency and the extension of the Old Aswan Hydropower stations (1 and 2) with total hydropower generation of about 10,000 GWh/year, which is about 23% of the total power generated in Egypt [Abu-Zeid and El-Shibini, 1997].

Figure 2.17. The AHD Power Station [Source: NWRP, 2005].

2.4.3.1.4 Irrigation

The vast increase of regulated water resources has permitted the cultivation of 10000 km^2 of new lands and the conversion of a further 4000 km^2 of basin irrigation in Upper Egypt to perennial irrigation with the possibility of cultivating two or three crops annually instead of one. The dam has made it possible to provide irrigation water for the various crops all year round and during the years with the least water, as well as ensuring the arrival of adequate quantities of water to the various crops at the right time. Accompanying measures include the improvement of drainage of all agricultural lands, thereby increasing productivity by approximately 20 percent as well as simplifying drainage networks and reducing their cost. Areas devoted to rice cultivation have increased from 1400 km^2 to about 4400 km^2 [Volker and Henry, 1997].

2.4.3.1.5 Land reclamation

The water secured by the long-term storage of the AHDR was the main factor that allowed the government of Egypt to fulfil its program of horizontal land expansion. About 0.84 million hectares were reclaimed, irrigated and cultivated using the water made available by the AHDR. This area encompassed lands in the East, West and Middle Delta and along the Nile Valley close to the old land. About half a million families were resettled on these new lands. The opportunity was then open for new employment and additional production, especially of foodstuffs of which Egypt is in dire need to cope with the increasing population and food consumption and to minimize food imports Egypt [Abu-Zeid and El-Shibini, 1997].

2.4.3.1.6 Navigation and river tourism

Navigation along the river has been improved, both upstream and downstream of the dam to the Mediterranean. This has resulted in an increase in the efficiency of transport economics. Recently, sailing along the Nile from Cairo to Aswan has become very popular and has attracted many groups of tourists from all over the world particularly in winter. The AHDR and its surroundings have become an area of interest and attraction to many tourists.

2.4.3.1.7 Fisheries and fish industries

Fisheries have developed rapidly in the AHDR with annual production of about 35000 tons [Abu-Zeid and El-Shibini, 1997]. Factories for the fishing industry and packaging are now in operation in the vicinity.

2.4.4 Egypt's Water Supply and Demands

2.4.4.1 Water resources availability

Egypt is located at the northern east corner of Africa where the arid climate is prevailing, and classified as the most arid country in the world with an average annual rainfall as low as 51 mm, which varies from 200 mm in Alexandria at the northern coast, 10 mm in Cairo, and almost zero in the inner areas of the western desert. Rainfall occurs only in winter season in the form of scattered showers [Attia, 2007].

The total annual available water resources in Egypt is estimated to be 57.7 BCM, as shown in figure 2.18, divided into 55.5 BCM of Nile water released from the AHDR, 1.3 BCM of effective rainfall, and 0.9 BCM of deep ground water. The percentage of these water resources to the total available water resources are 96.2% for Nile water, 2.2% for effective rainfall, and 1.6% for desert deep groundwater [MWRI, 2005]. The per capita of available water resources in year 2000 was 859 m^3, less than the water poverty limit, which is specified as1000 m^3/year, and expected to decrease to 720 m^3/year by the year 2017.

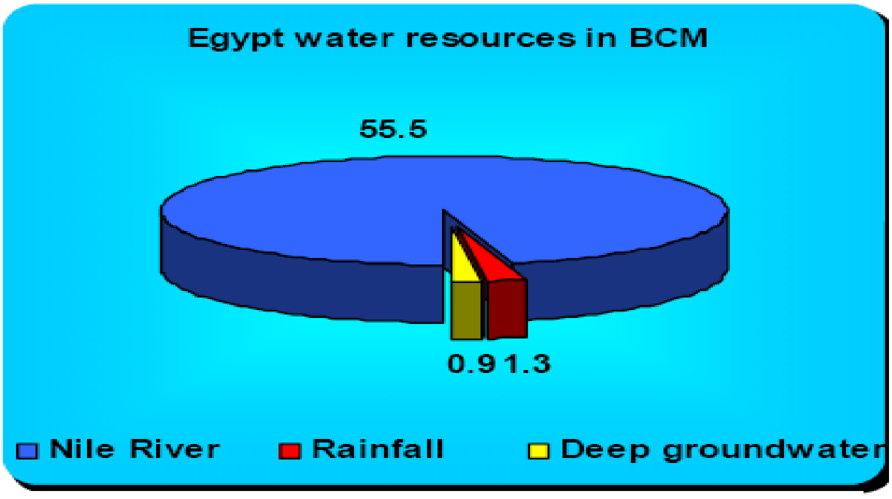


Figure 2.18. Egypt's Water Resources [Source: MWRI, 2005].

2.4.4.1.1 Surface water

Egypt receives about 98 % of its fresh water resources from outside its internationals borders. This is considered to be a main challenge for water policy and decision makers in the country as the river provides the country with more than 95% of its various water requirements. Water released from the AHD is distributed among the whole country through a canal system (figure 2.19) consisted of major canals that divert water to lower order canals such as branch canals, lateral canals, distributary canals, and field watercourses.

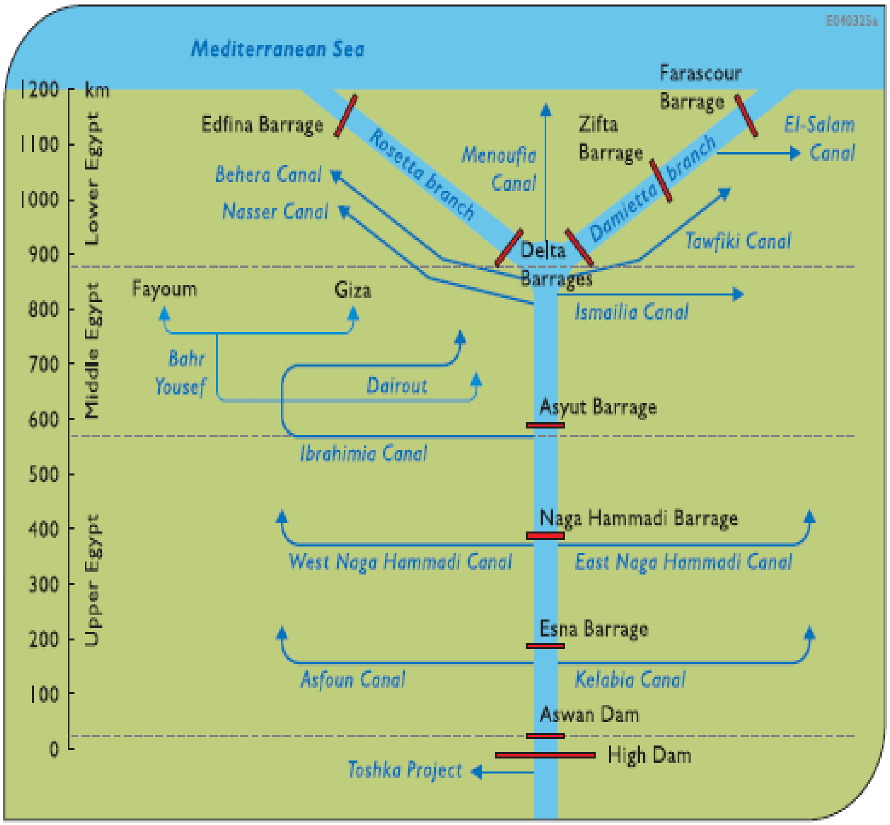


Figure 2.19. Schematic diagram of major control structures on the Nile in Egypt [Source: MWRI, 2005].

This large extensive irrigation network deliver water mainly under gravity through a large number of regulators, weirs, and other hydraulic structures. Beside the gravity diversion of water, there are also more than 100 major pumping stations along the Nile and its branches [MWRI, 2005].
Figure 2.20 illustrates a typical water distribution through the Nile System for a Nile discharge of 55 BCM/yr. It shows the order of magnitude only.

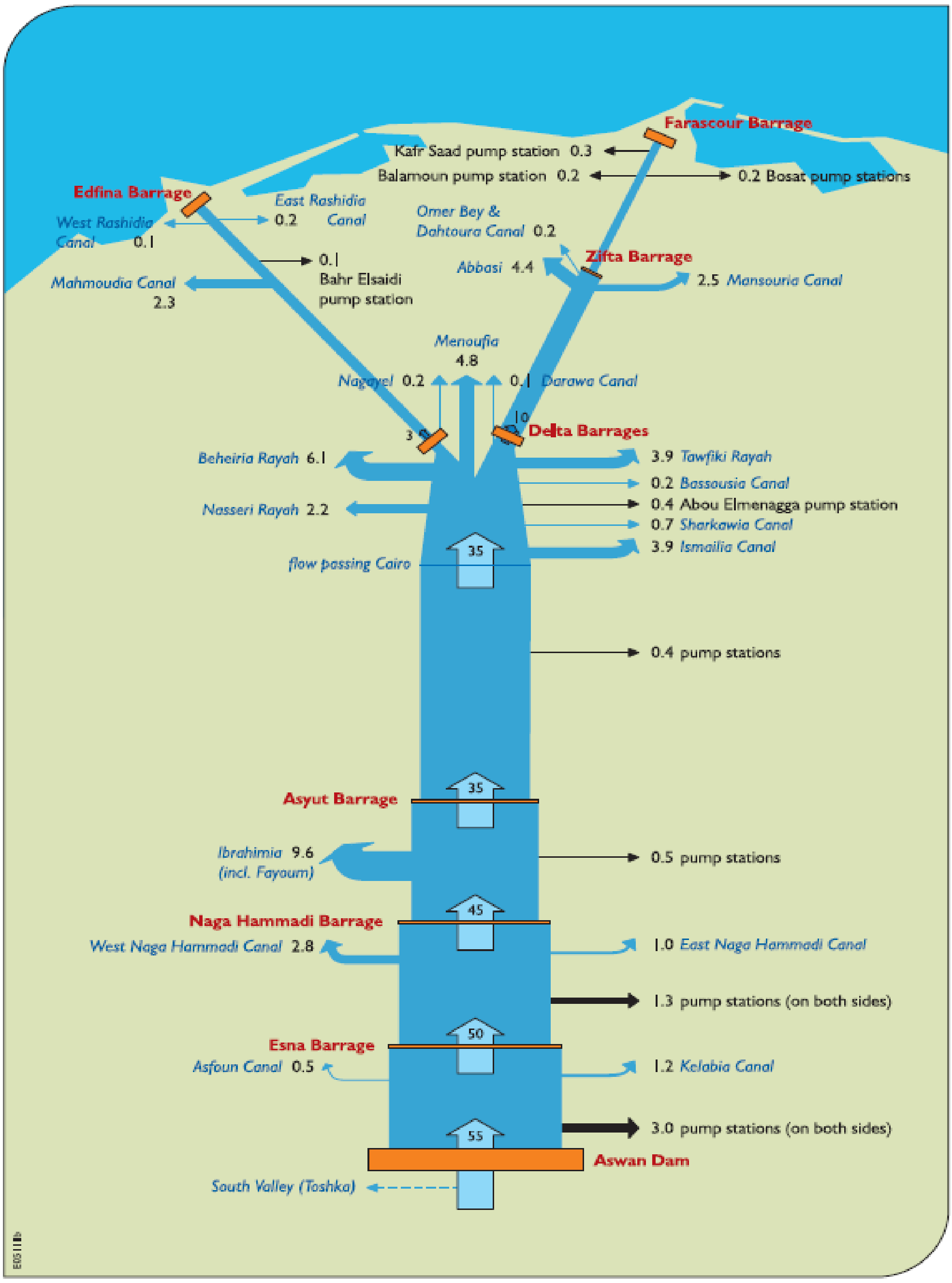


Figure 2.20. Water dstribution Nile system [Source: MWRI, 2005].

Beside the irrigation network, there is also a huge drainage network, which carries the water drained from the agricultural lands and also effluents from municipalities and industries. This system starts at field drains (open or subsurface) then collector drains and main drains, which deliver water either back to the Nile, costal or inland lakes, or directly to the sea. This delivery depends mainly on gravity except for a number of pumping station in Northern Delta. The drainage network carries annual discharge of about 17 BCM, from which 4 BCM are reused (officially) and 13 BCM are delivered to the sea or lakes. Most of the Upper Egypt drains discharge water into the Nile, while most drains in Delta discharge water into northern lakes or directly to the sea [MWRI, 2005].
The distribution of Nile water quality is nearly uniform from Aswan to Cairo. Total Dissolved Solids (TDS) ranges from 130 parts per million (ppm) at the AHDR to 200-250 ppm at Cairo, but increases in the two Nile branches towards north up to 500 ppm, because they receive nutrients, organic loads, grease, and oils. The power of hydrogen (PH) increases from 7.7 at Aswan to 8.5 in Delta. Mostly the dissolved oxygen does not go below 5 milligrams per litter. Nitrate and ammonium hardly exceed the current standards. That is due to self-purification of the river [MWRI, 2005].

2.4.4.1.2 Groundwater

Although in terms of quantity the contribution of groundwater to the total water supply in Egypt has been very moderate, groundwater is the sole source of water for people living in the desert areas. Because of limited options to increase the Nile water availability, there has been an increasing interest during the last decade to further develop the groundwater resources. The aquifers carrying groundwater in Egypt can be classified as following (see figure 2.21) [MWRI, 2005]:

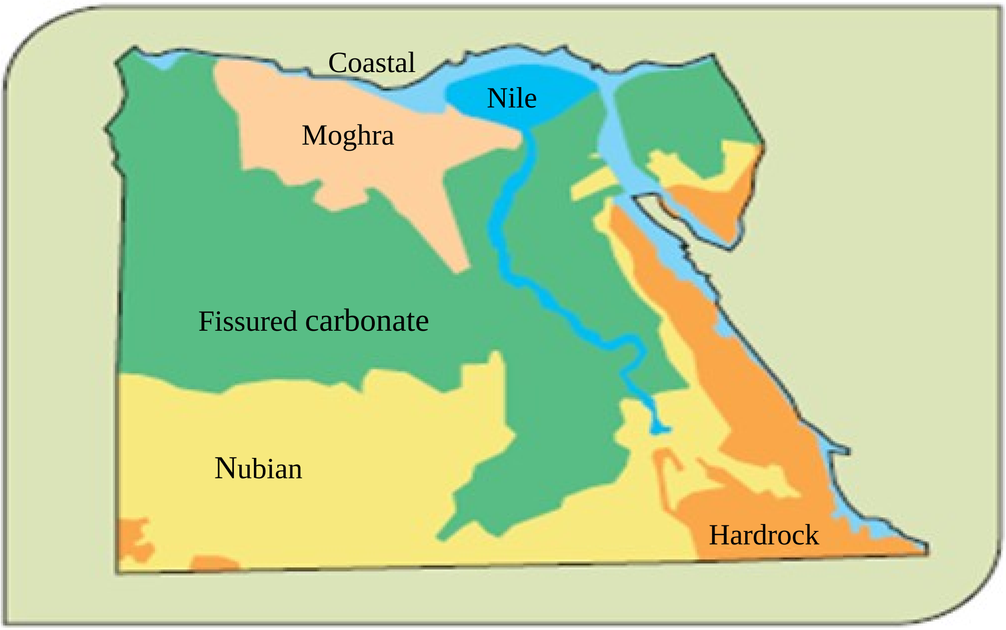


Figure 2.21. The major aquifer systems in Egypt [Source: MWRI, 2005].

Nile aquifer: Although this aquifer supplies the irrigation system with more than 6 BCM every year (87% of groundwater abstraction), it is not an independent source of water. It is recharged by the Nile water seepage and deep percolation of the irrigated lands. The water table depth may be of few meters below ground surface that allows low pumping cost and relatively large quantities of water abstraction (30-100 m^3/hr/well). During period of peak irrigation, farmers may use groundwater conjunctively with surface water. Water quality is still fairly good. However, 20% of water does not meet the standards of drinking water. Total Dissolved Solids (TDS) is less than 1000 parts per million (ppm) in the upper zone of the aquifer, while brackish water (up to 5000 ppm) is found in the lower zone of the aquifer, in the fringe of the valley, and the northern half of the Delta.
Nubian sandstone aquifer: It is considered as the most important groundwater body, which covers an area of 2 million km^2 and extends into Sudan, Libya, and Chad. The thickness of bearing layer is 200-3500 m and the total volume of stored fresh water may be 150,000 BCM of fossil and non-renewable water. The groundwater table may go up to 2000 m below ground surface. So that, the development of large areas is restricted. The water quality is usually very good. The salinity of the fresh part of water varies both vertically and horizontally. South of the latitude of Bani-Suif (29° N) the salinity ranges between 100–500 ppm. In Kharga and Dakhla oasis's, the upper layers salinity is 1000 ppm and decreases in deeper layers to 200 ppm.
Fissured carbonate aquifer: It covers 50% of Egypt area, and acts as a confining layer of the Nubian sandstone aquifer. The aquifer recharge is very limited and no reliable figures are available about its potential. In Siwa oasis, a well can abstract 5-300 m^3/hr. The aquifer contains brackish water. Fresh water is found only where the aquifer is recharged through infiltration from wadis or seepage from the underlying Nubian sandstone aquifer.
Moghra aquifer: The aquifer is recharged by rainfall and lateral inflow from the Nile aquifer. It contains fresh water only near its eastern border, but the salinity increases rapidly toward the north and west. As the aquifer is located by the west fringe of the Nile delta, it is subjected to heavy abstraction, and hence, the water quality is deteriorated and its levels dropped greatly.
Coastal aquifer: These aquifers extend along northern and western coasts and are recharged by rainfall. The benefit of these aquifers is limited due to salt intrusion.

2.4.4.1.3 Desalination of seawater

Egypt has about 2,400 km of shorelines on both the Red Sea and the Mediterranean Sea. Therefore, desalination can be used as a sustainable water resource for domestic uses in many locations. This is actually practiced in the Red Sea coastal area to supply tourism villages and resorts with adequate domestic water supply where the economic value of the unit of water is high enough to cover the cost of desalination.

2.4.4.1.4 Non-conventional water resources

- Reuse of drainage water

The agricultural drainage water of the southern part of Egypt returns directly to the Nile River where it is mixed automatically with the Nile fresh water to be used for different purposes in the downstream. The total amount of official reuse of agricultural drainage water was estimated to be

5.96 BCM in 99/2000. Reuse of agricultural drainage water is limited by the salt concentration of the drainage water. Therefore, more efficient irrigation, inevitability, leads to the same amount of salt dissolved in a smaller volume of drainage water. A major problem experienced, in drainage water reuse, is the deteriorating water quality in many drains that are polluted from municipal and industrial sources. Mixing of this water with canal water in a number of cases threatened other water users that are located downstream of the mixing points. Large efforts to reduce the pollution loads should be exerted.
As an alternative to the reuse of drainage water from larger drains, the reuse could shift to smaller less polluted drains in the upper part of the system. This so-called intermediate reuse would pump drainage water to lower order irrigation canals where it does not have harmful impacts on downstream domestic water intakes.

- Reuse of treated wastewater

Primary use of treated wastewater is of irrigation of green areas (landscape development) and irrigation of non-food agriculture. The ministry of Environment in cooperation with MALR and MHUNC is executing a national program for reuse of treated wastewater in forestation.

2.4.4.2 Present and future water demands

There are many water-related challenges facing Egypt. The most important challenge is Egypt's expected population growth: from 63 million in 2000 to 83 million in 2017) and related water demand for public water supply and economic activities, in particular agriculture. To relieve the population pressure in the Nile Delta and Nile Valley, the government has embarked on an ambitious program to increase the inhabited area in Egypt (from 5.5% living outside the Nile Valley and Delta to about 25%). Industrial growth, the need to feed the growing population and hence a growing demand for water by agriculture, horizontal expansion in the desert areas, etc. cause a growing demand for water. At the same time the available fresh water resources are expected to remain more or less the same. This urges to make a more efficient use of present resources and, if possible, to develop additional Cairo; a city of millions and still growing water resources.

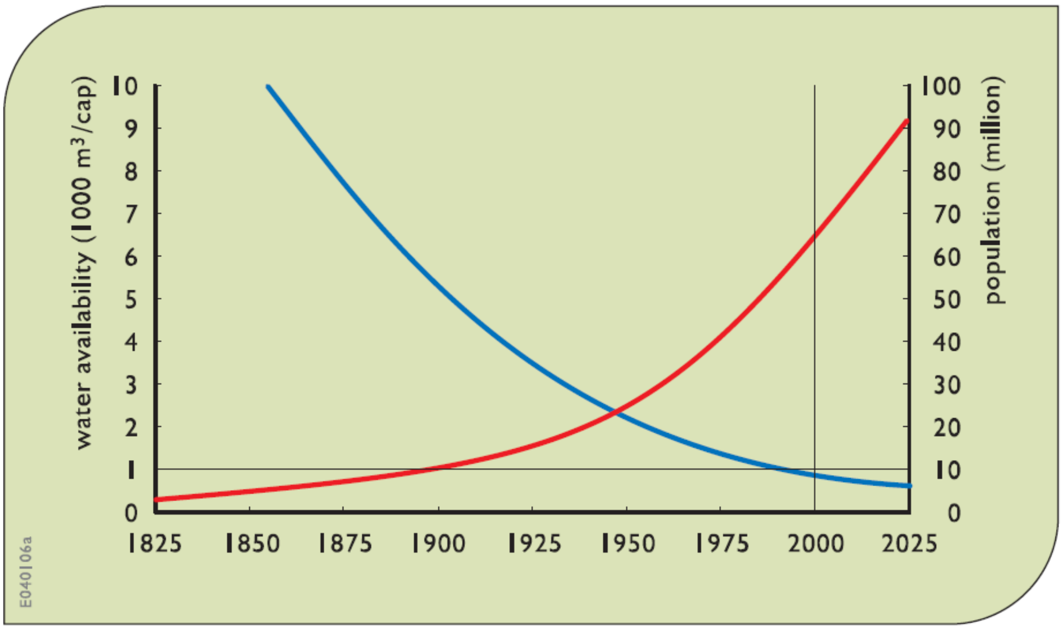


Figure 2.22. Population growth and water availability [Source: MWRI, 2005].

As a rough global indicator of water sufficiency the annual amount of water available per capita is often mentioned in the literature. This amount includes water for all purposes, including water for food production. If less than 1000 m^3 per capita per year is available, water scarcity occurs. In Egypt this critical value was reached around 1997 as indicated in figure 2.22.
Due to further population increase the per capita amount of water is expected to decrease to 720 m^3 per year in 2017. Although the 1000 m^3 criterion for water scarcity may be debatable, it seems safe to conclude that water is becoming a scarce commodity by the year 2017 [MWRI, 2005].

2.4.4.2.1 Present water demands

Egypt's water requirements increase with time due to the increase in population and the improvement of living standards as well as the government policy to reclaim new lands and encourage industrialization. Water demand was estimated to be 68.7 BCM in year 2000 (see figure 2.23) [Attia, 2007], i.e., there was water shortage of about 18% of the available water resources. To overcome this shortage, part of the agricultural drainage is reused, beside the use of shallow groundwater and non-conventional resources. The following is a description of the main requirements.

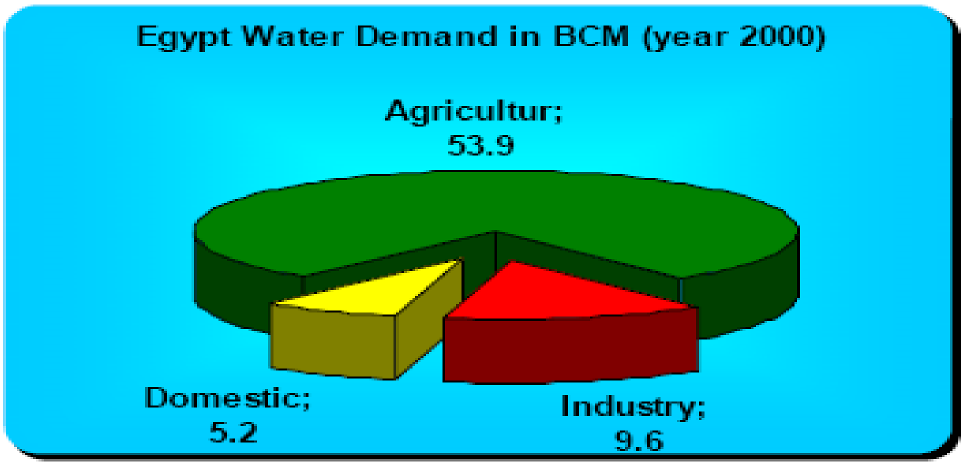


Figure 2.23. Egypt water demand in BCM (Year 2000) [Source: Attia, 2007].

2.4.4.2.1.1 Agriculture water requirements

The agricultural sector is the largest user, and consumer, of water in Egypt, with its share exceeding 78 % of the total demand for water. Therefore, most land and water policies are mostly concerned with agriculture. The plan for agricultural horizontal expansion of cultivated land is considered a national plan aiming to increase the agriculture land and crop production. Therefore, cultivated and cropped areas are increasing in the past few years (cultivated area in 1990 was only 2.9 million hectares, while the cropped area was 5.22 million hectares). The total diverted water to agriculture including conveyance and application losses, in 2000 was about 53.90 BCM.

2.4.4.2.1.2 Municipal water requirements

The total municipal water use was estimated to be 5.2 BCM in year 2000 [Attia, 2007]. A portion of that water is actually consumed and the rest returns back to the system, either through the sewage collection system or by seepage to the groundwater. This water is delivered to the users through municipal distribution networks in urban areas and few villages. The major factor affecting the amount of diverted water for municipal use is the efficiency of these delivery networks.

2.4.4.2.1.3 Industrial water requirements

In 1990, the general authority for industry made a survey that covered 90% of the public sector major factories to estimate industrial needs and requirements. The results of the study were used to estimate the water requirement for the industrial sector during the year 2000 where the estimated value was 9.6 BCM/year. A small portion of the diverted water for industrial requirement is consumed through evaporation during industrial processes while most of that water returns back to the system. The study of the Water Master Plan suggested that only about 6% of water abstracted by industry is consumed which means only 0.45 BCM of water delivered to industry in year 2000 was lost. Thus, a huge volume of partially treated or untreated effluent is returning to the system creating major environmental problems.

2.4.4.2.2 Future water demands

The total area of irrigated land in the year 2000 was approximately 3.25 million hectares and expected to be 4.6 million hectares by the year 2017 due to horizontal expansion and the implementation of the two mega projects of El-Salam Canal at North Sinai and Toshka at south valley. Consequently, the agriculture demand is expected to increase from 53.9 to 63.6 BCM (figure 2.24) taking into consideration the rising of irrigation efficiency by extending the irrigation improvement projects to cover most of the old lands, and applying modern irrigation techniques, e.g., sprinkler and drip irrigation, in the new reclamation lands.
To estimate the increase of municipal (domestic) water demand until year 2017, the increase of per capita income is assumed 4.3 % per year, then the annual per capita increase in domestic water demand is taken as 0.1 of the per capita income increase, i.e., 0.43 % per year, which is corresponding to an increase of 7.6 % between years 2000 and 2017. As the population was 68 millions in the year 2000 and expected to reach 83 millions in the year 2017, then the domestic water demand in 2017 may go up to 127 % of the demand of year 2000, i.e., about 6.6 BCM.
To estimate the industrial water demand for year 2017, it is assumed that the demand of old industries will increase by 20%, and the new industries will increase the industrial area from 102 km^2 in year 2000 to 305 km^2 in year 2017, the design supply is about 7000 m^3/km^2/day. Taking into consideration the increase of demand in mining areas, the total industrial demand may increase from 9.6 BCM in the year 2000 to 13.5 BCM in the year 2017 [MWRI, 2005].

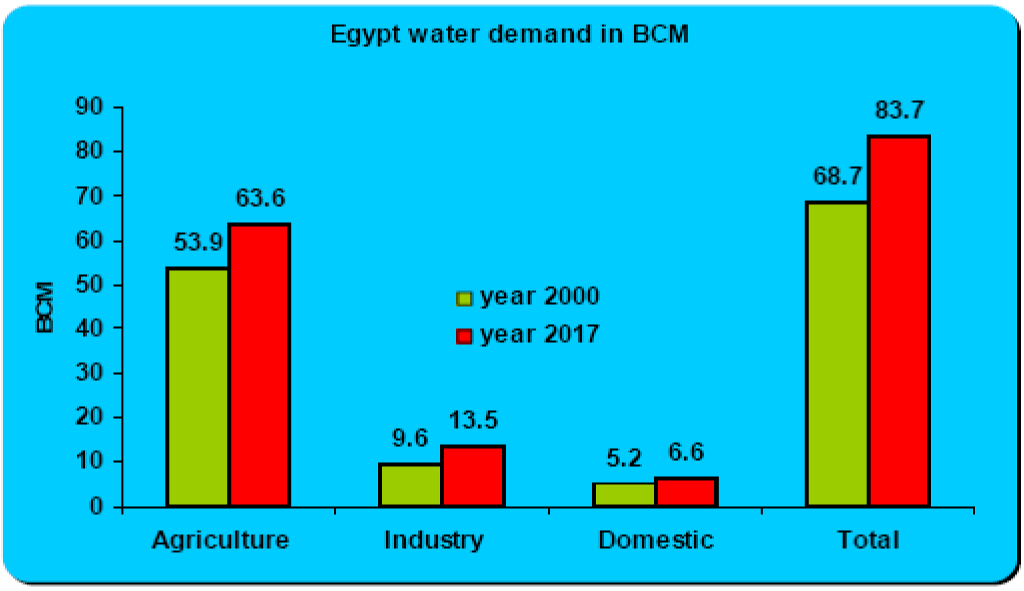
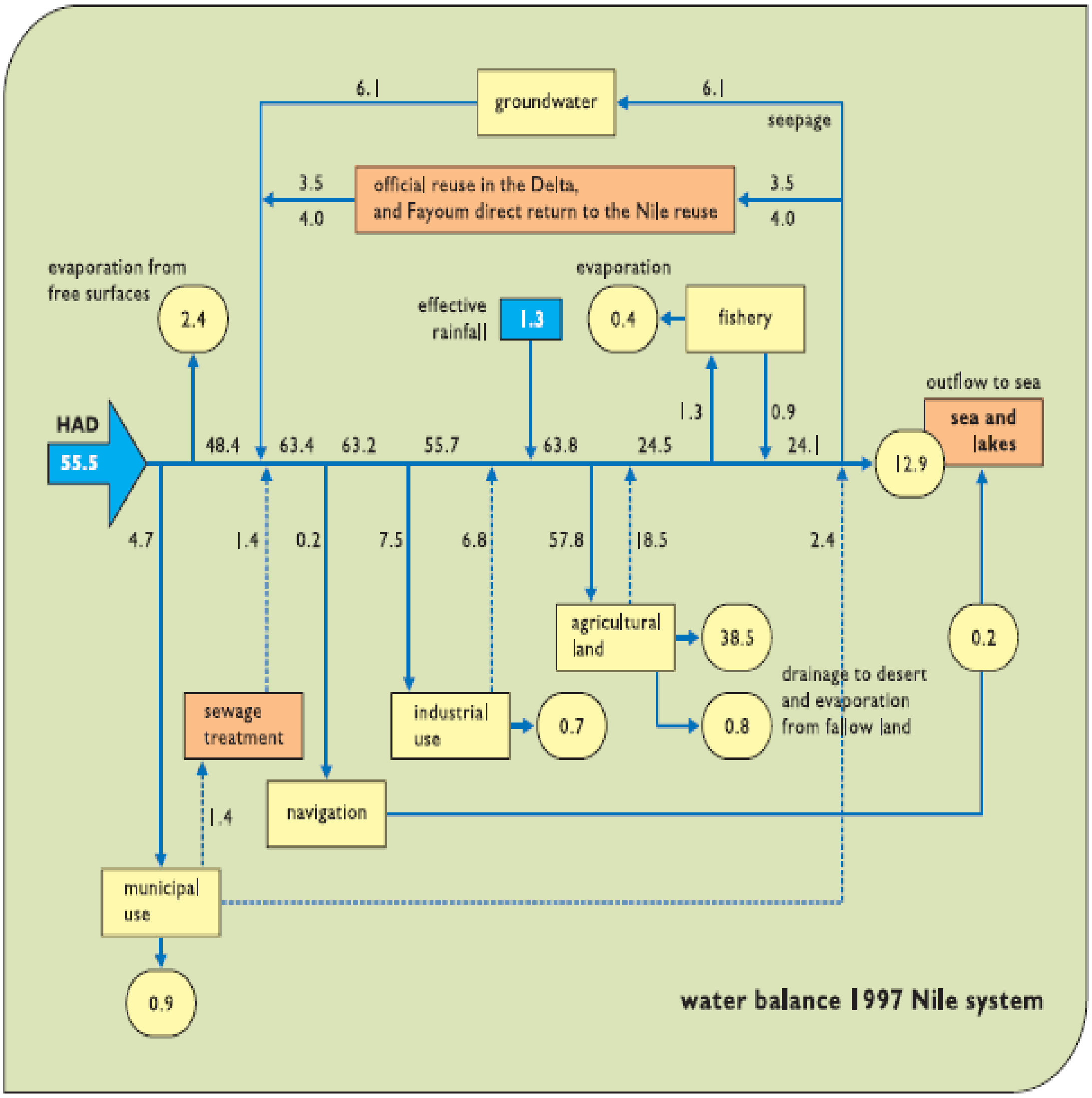


Figure 2.24. Current and future water demand (year 2000 and 2017) [Source: MWRI, 2005].

2.4.4.3 Water balance of Egypt

On its route from the AHDR to the Mediterranean the water of the Nile River is re-used several times. In the Valley water is abstracted from the river for irrigation. Part of that water is returned to the river as drainage water and can be used again downstream. The same applies to the abstractions for drinking water and industrial water. This reuse of the water makes the water balance of Egypt quite complex.

In fact, some of the users ‘consume’ only a fraction of the water they withdraw. The remainder is discharged back to the system. Examples are the Municipal Use that consumes only 0.9 BCM of their water withdrawal of 4.7 BCM in 1997, and Fishery that consumes (evaporates) only 0.4 BCM of their demand of 1.3 BCM. The water balances of Egypt for 1997 and 2017 are given in figures 2.25 and 2.26. These water balances provide some more detail and show the gross demand of the various uses and their return flow [MWRI, 2005].



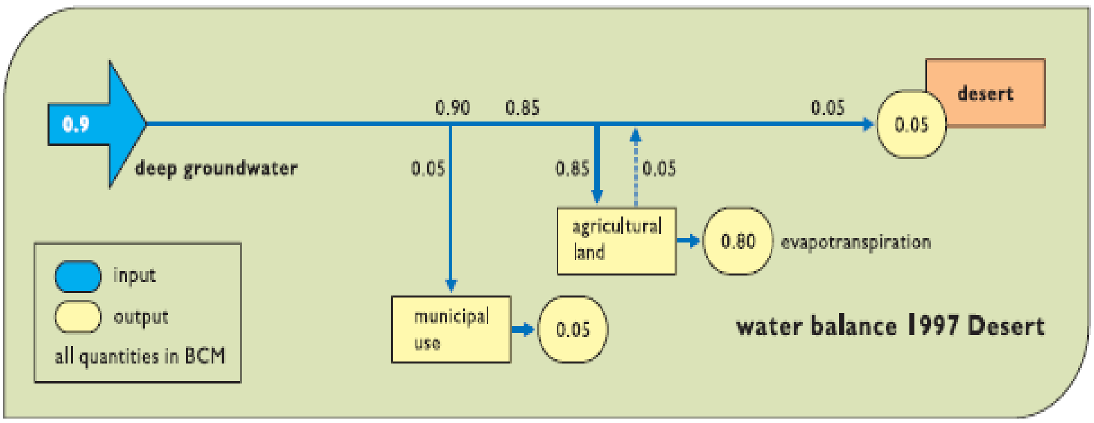


Figure 2.25. Water balance of Egypt 1997 [Source: MWRI, 2005].

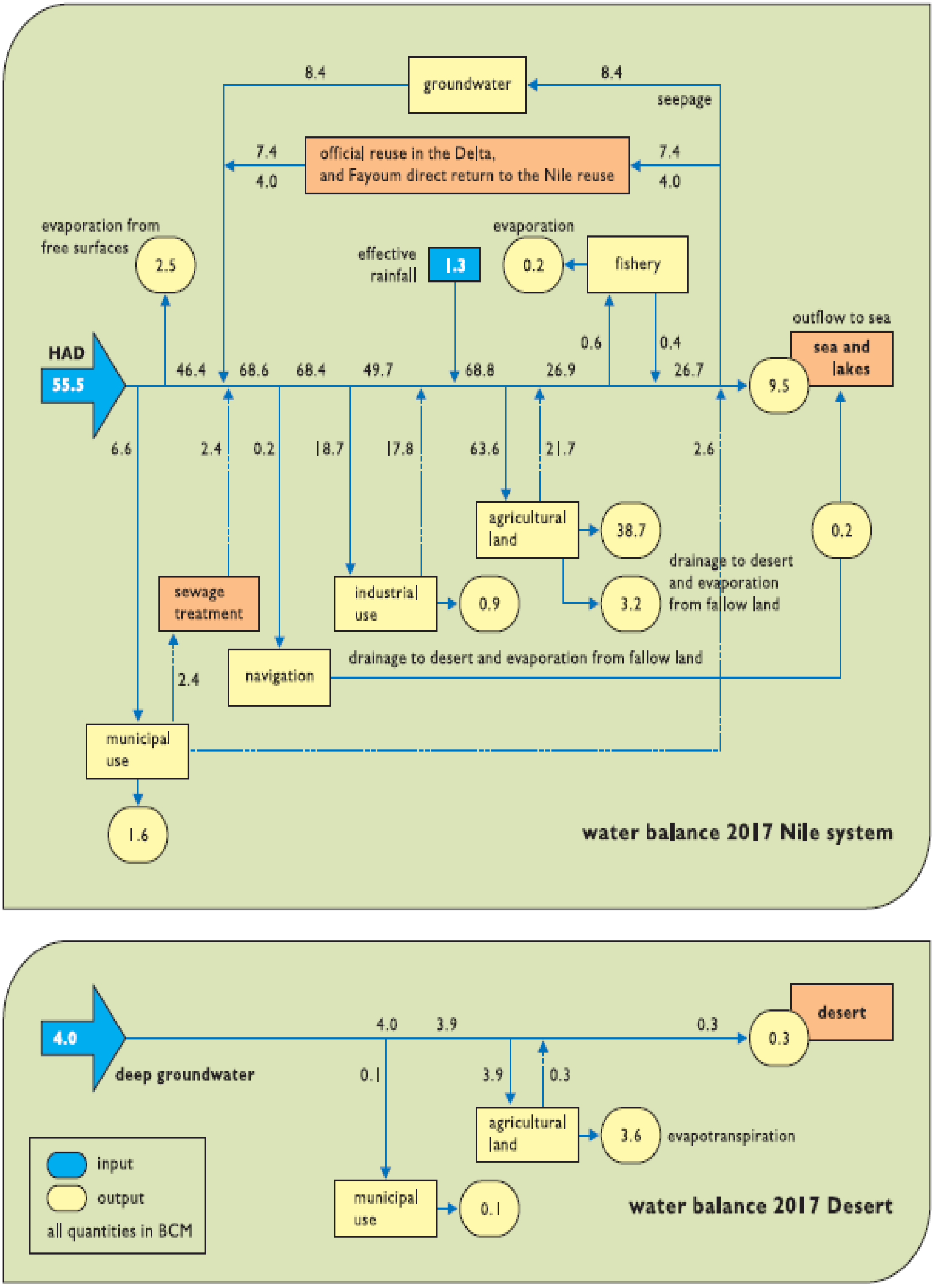


Figure 2.26. Water Balance of Egypt 2017 [Source: MWRI, 2005].

3 METHODOLOGY

3.1GENERAL

Reservoir operation is a complex problem that involves many decision variables, multiple objectives as well as considerable risk and uncertainty [Oliveira and Loucks, 1997]. Operating policies differ from case to case primarily because of the different social, economic, and political objectives that a water resources system is supposed to attain. In addition, the conflicting objectives lead to significant challenges for operators when making operational decisions.
Traditionally, reservoir operation is based on heuristic procedures, embracing rule curves and subjective judgments by the operator. This provides general operation strategies for reservoir releases according to the current reservoir level, hydrological conditions, water demands and the time of the year. Figure 3.1 shows the different elements for the multipurpose reservoir systems [Ostrowski, 2009].

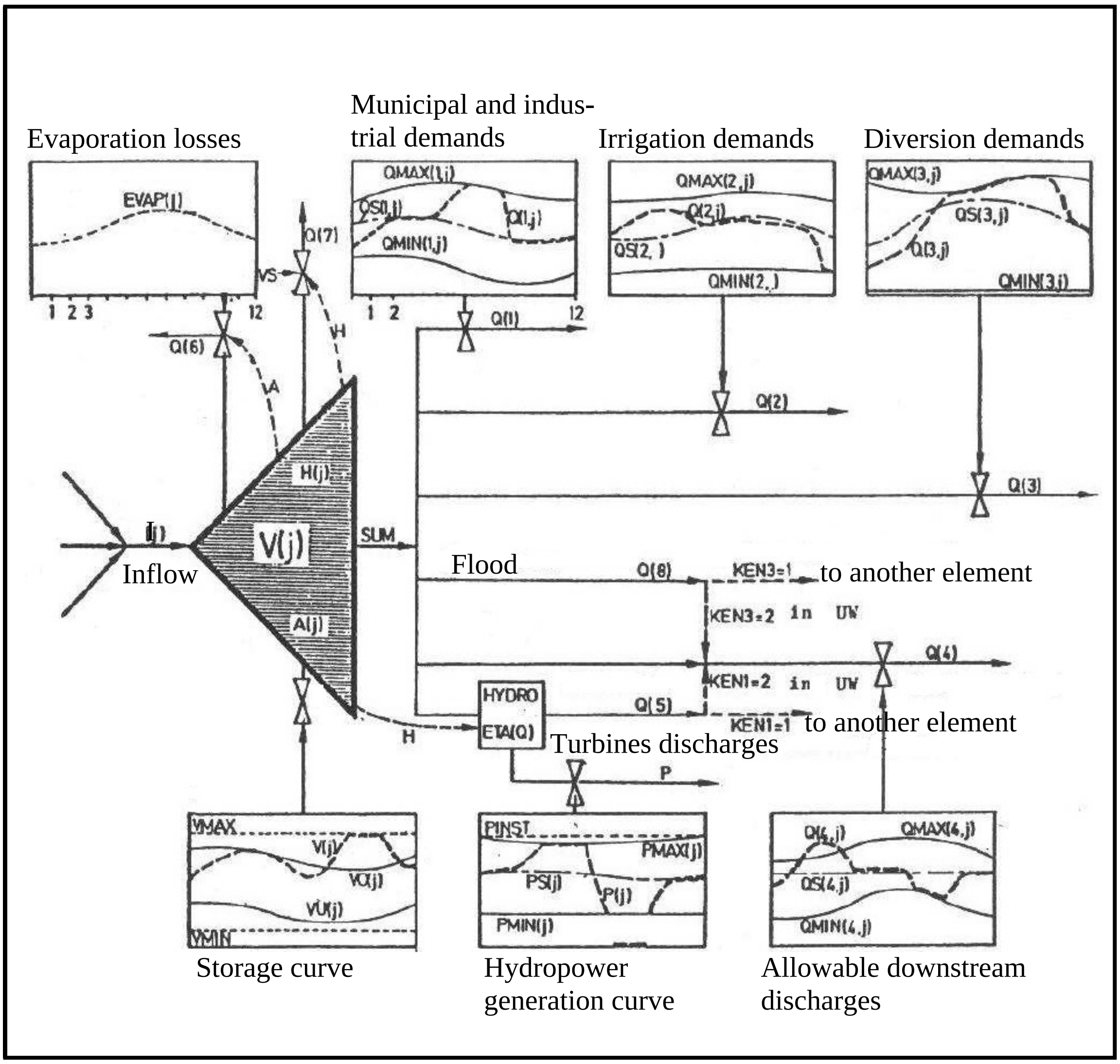


Figure 3.1. Multipurpose reservoir system [Ostrowski, 2009].

3.2 PROPOSED METHODOLOGY

In this study the AHDR is modeled using the concept of piecewise linearization [Ostrowski, 2010]. According to Ostrowski, The piecewise linearised analytic approach offers the opportunity to determine multiple processes being nonlinear functions of storage content for a chosen time arbitrary interval without excessive numerical iterations. By combination of external known or assumed stochastic events, the relationship between process und storage can be scaled. This also facilitates the simulation of operation and control rules.

For the concept of piecewise linearization, a storage element loaded with several time series Z_i and processes $P_j(V(t))$ is shown in figure 3.2.

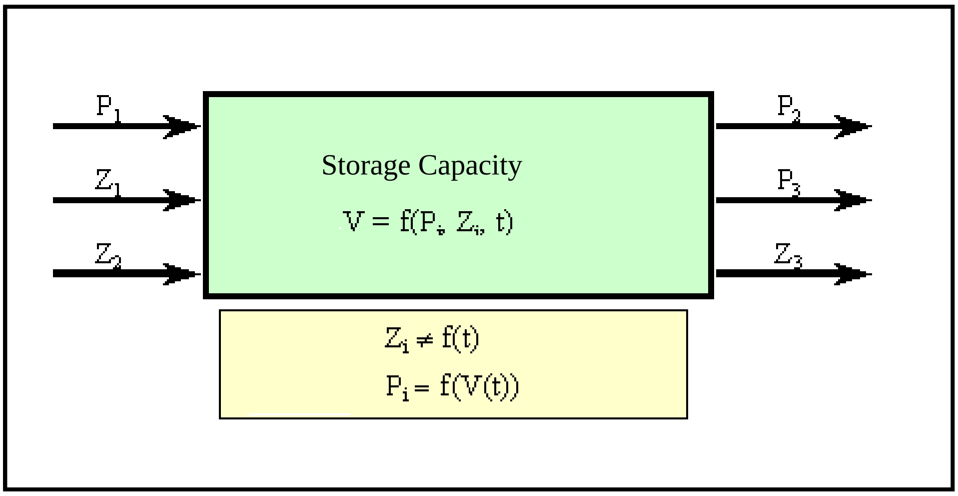


Figure 3.2. A storage element with several time series and processes [Ostrowski, 2010].

The continuity equation for such an element is written as:

$$\frac{dV(t)}{dt} = Z_1 + Z_2 - Z_3 + P_1(V(t)) - P_2(V(t)) - P_3(V(t)) \quad (3.1)$$

The functional relationships are not explicitly given in the following equations, as the notation chosen is clear.

The Z_i terms are functions of time, but are considered constant during a single computational time interval. Thus, they can be summarised, which leads to Eq. 3.2.

$$\frac{dV(t)}{dt} = \sum_{i=1}^{n_Z} Z_i + P_1 - P_2 - P_3 \quad (3.2)$$

In figure 3.3 two processes P1 (V) and P2 (V) are given, replacing continuous nonlinear relationships by polygons deviding the storage element into arbitray reaches. The slope of the linearised functions is defined by as $m_{j,k}$. Each process can now be defined according to Eq. 3.3.

$$P_j(V(t)) = P_{j,k-1} + m_{j,k} \cdot (V(t) - V_{j,k-1}) \tag{3.3}$$

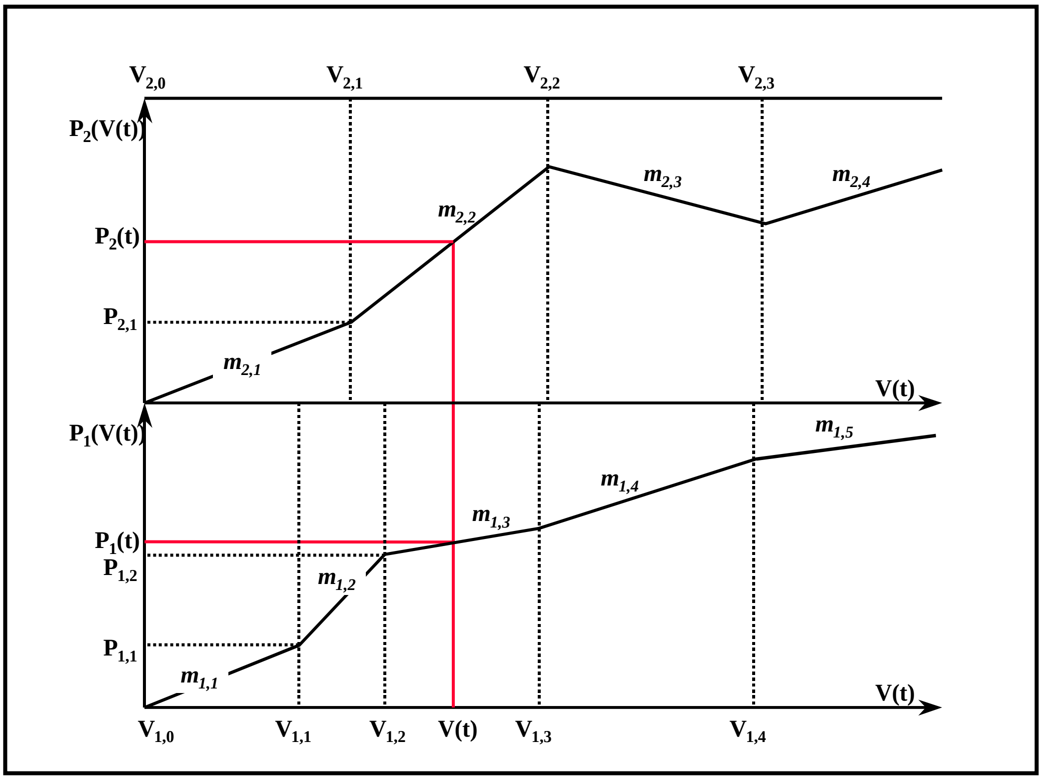


Figure 3.3. Identification of two process variables P_1 (t) and P_2 (t) for a storage element [Ostrowski, 2010].

By dissolving the brackets two terms can be defined one being independent and the other independent of V(t) according to Eq. 3.4:

$$P_j(V(t)) = \underbrace{\left(P_{j,k-1} - m_{j,k} \cdot V_{j,k-1}\right)}_{constant} + \underbrace{m_{j,k} \cdot V(t)}_{f(t)} \tag{3.4}$$

The dependent and independent j process terms are then summarised according Eq. 3.5:

$$\sum_{j=1}^{n_P} P_j = \sum_{j=1}^{n_P} \left(P_{j,k-1} - m_{j,k} \cdot V_{j,k-1}\right) + \sum_{j=1}^{n_P} m_{j,k} \cdot V(t) \tag{3.5}$$

Inserting Eq. 3.5 into the continuity equation results in Eq. 3.6:

$$\frac{dV}{dt} = \underbrace{\sum_{i=1}^{n_Z} Z_i + \sum_{j=1}^{n_P}\left(P_{j,k-1} - m_{j,k}\cdot V_{j,k-1}\right)}_{C1} + \underbrace{\sum_{j=1}^{n_P} m_{j,k}\cdot V(t)}_{C3} \tag{3.6}$$

Except for the + of the dependent term the equation is identical to the continuity equation of the linear reservoir ($dV(t)/t=Q_{inflw}-k.V(t)$). By setting C3 = -C2 and subsequent substitution we arrive at Eq. 3.7:

$$\frac{dV}{dt} = C1 - C2\cdot V(t) \tag{3.7}$$

With known solution in Eq. 3.8:

$$V(t) = \frac{C1}{C2}\cdot\left[1 - e^{-C2\cdot(t-t_0)}\right] + V_0\cdot e^{-C2\cdot(t-t_0)} \tag{3.8}$$

As a convention, it is assumed that time series as well as processes are defined as Inflow ⇔ positive und Outflow ⇔ negative. By summing up the single terms it is determined automatically whether the storage element is filled or emptied.
To determine the process intensity at a certain time t Eq. 3.8 can be solved for V(t), which is then inserted into Eq. 3.1 to compute $P_j(V(t))$. The same problem occurs as in the case of the linear reservoir concerning the determination of mean values during the computational time interval, i.e. assuming linear processes might lead to relevant errors. Therefore the procedure described above for the simple case, is demonstrated again for the multiple input/output reservoir.

$$P_j(V(t)) = \left[\frac{C1}{C2}\cdot\left[1 - e^{-C2\cdot(t-t_0)}\right] + V_0\cdot e^{-C2\cdot(t-t_0)} - V_{j,k-1}\right]\cdot m_{j,k} + P_{j,k-1} \tag{3.9}$$

$$\overline{P_j(t_0 \to t_1)} = \frac{1}{t_1 - t_0}\cdot\int_{t_0}^{t_1} P_j(V(t))\, dt \tag{3.10}$$

The solution being Eq. 3.11:

$$\overline{P_j} = \left[\frac{C1}{C2}\cdot\left[t + \frac{1}{C2}\cdot e^{-C2\cdot(t-t_0)}\right] - \frac{V_0}{C2}\cdot e^{-C2\cdot(t-t_0)} - V_{j,k-1}\cdot t\right]\cdot m_{j,k} + P_{j,k-1}\cdot t\Bigg|_{t_0}^{t_1} \tag{3.11}$$

including integration boundaries t_0 and t_1 and finally summarising the terms, the solution becomes Eq. 3.12:

$$\overline{P_j} = P_{j,k-1} - m_{j,k} \cdot V_{j,k-1} + \\ m_{j,k} \cdot \left[\frac{C1}{C2} + \left[1 - e^{-C2 \cdot (t_1 - t_0)}\right] \cdot \left[\frac{V_0}{(t_1 - t_0) \cdot C2} - \frac{C1}{(t_1 - t_0) \cdot C2^2} \right] \right] \tag{3.12}$$

Time t_1 in Eq. 3.12 is either the end of the external computational time interval chosen or the internal time to reach the boundaries of a linear function increment. Time t_1 when a boundary is reached can be determined according to Eq. 3.12, by setting V(t) equal to $V_{j,k-1}$ when the content decreases or equal to $V_{j,k}$ when the content increases.

$$t_1 = -\frac{1}{C2} \cdot \ln \left[\frac{V(t) - \frac{C1}{C2}}{V_0 - \frac{C1}{C2}} \right] + t_0 \tag{3.13}$$

Whether the content is increasing or decreasing during a time interval can be identified from Eq. 3.12. First it is assumed that the content is increasing and thus V(t) becomes $V_{j,k}$. The value of t1 defines three different cases:

1. t1 > Δt

   In the time step no boundary limit is reached, Eq. 3.11 can be applied

2. 0 < t1 < Δt
   After time t_1 the upper reach boundary is exceeded, a change to the next reach is required computing new constants

3. t1 < Δt
   The assumption of increasing storage content was wrong. The new boundary is the the lower value $V_{j,k-1}$ instead of $V_{j,k}$ as the sum of total outflow components is smaller than the sum of total outflow components.

As each process can have its own discretisation, it is necessary to determine the adequate $V_{j,k}$ before solving Eq. 3.12. being the nearest one to V(t).
The relevant actual time interval $\Delta t_{relevant}$ is determined according to Eq. 3.14.

$$\Delta t_{relevant} = \min\left[(t - t_0), (t_{1,j} - t_0)\right] \tag{3.14}$$

$\Delta t_{relevat}$ is used in Eq. 3.14 to compute mean process rates. Unless the outer loop for the total time interval is finished constants C1 and C2 have to be updated according to Eq. 3.7 for the next time increment. Volume changes are computed for the total time interval by integration of all mean process rates P_j und external inflows and outflows Z_i.

3.2.1 Consideration of Independent Variable External Inflows and Outflows as f(t)

The storage concept explained above provides high flexibility for the formulation of transfer functions being dependant on the storage content as a function of time. In hydrology und water resources processes can be a function both of storage and of external events. Below the concept is further extended to account for this relevant aspect.
According to Eq 3.15 a process can be defined as the product of a standardised process function FP_j being dependent on storage and an external time series ZP_j.

$$P_j(V(t),t) = ZP_j(t) \cdot FP_j(V(t)) \tag{3.15}$$

According to Figure 3.4 the actual value of $P_j(V(t))$ is determined according to Eq. 3.16 between times t2 and t3:

$$P(V(t),t) = ZP_3 \cdot [FP_2 + m_3 \cdot (V(t) - V_2)] \tag{3.16}$$

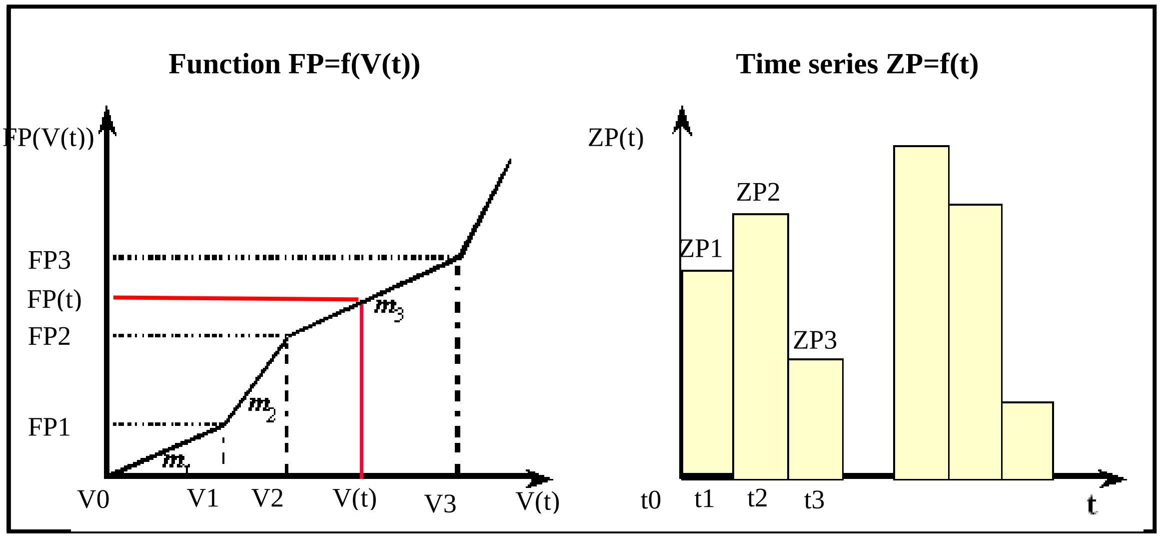


Figure 3.4. A standardised process function and time series [Ostrowski, 2010].

or generally defined in Eq. 3.17:

$$P_j(V(t),t) = ZP_m \cdot \left[FP_{j,k-1} + m_{j,k} \cdot (V(t) - V_{j,k-1})\right] \tag{3.17}$$

In Eq 3.18 again dependent und independent terms are determined:

$$P_j(V(t),t) = ZP_m \cdot FP_{j,k-1} - ZP_m \cdot m_{j,k} \cdot V_{j,k-1} + ZP_m \cdot m_{j,k} \cdot V(t) \tag{3.18}$$

Even more generally Eq. 3.19 accounts for multiple n_p processes :

$$\sum_{j=1}^{n_P} P_j(V(t),t) = \sum_{j=1}^{n_P}\left(ZP_m \cdot FP_{j,k-1} - ZP_m \cdot m_{j,k} \cdot V_{j,k-1}\right) + \sum_{j=1}^{n_P}\left(ZP_m \cdot m_{j,k}\right) \cdot V(t) \tag{3.19}$$

in combination with the continuity equation we arrive at Eq. 3.20 with know solution:

$$\frac{dV}{dt} = \underbrace{\sum_{i=1}^{n_Z} Z_i + \sum_{j=1}^{n_P}\left(ZP_m \cdot FP_{j,k-1} - ZP_m \cdot m_{j,k} \cdot V_{j,k-1}\right)}_{\mathrm{C1}} + \underbrace{\sum_{j=1}^{n_P}\left(ZP_m \cdot m_{j,k}\right)}_{\mathrm{C3}} \cdot V(t) \tag{3.20}$$

Mean process rates are finally computed with Eq. 3.21 and Eq. 3.22:

$$\overline{P_j} = ZP_m \cdot \left[\begin{array}{l} FP_{j,k-1} - m_{j,k} \cdot V_{j,k-1} + \\ m_{j,k} \cdot \left[\dfrac{C1}{C2} + \left[1 - \mathrm{e}^{-C2 \cdot (\Delta t_{ma})}\right] \cdot \left[\dfrac{V_0}{(\Delta t_{ma}) \cdot C2} - \dfrac{C1}{(\Delta t_{ma}) \cdot C2^2}\right]\right] \end{array}\right] \tag{3.21}$$

with:

$$\Delta t_{ma} = \min\left[(t - t_0), (t_{1,j} - t_0)\right] \tag{3.22}$$

3.3 BLUEM - STRUCTURE OPERATION MODULE

In this study BlueM will be used to analyses future development of water resources yield and demand and related potential future modifications of the infrastructure and its operation strategies for the AHDR.

BlueM, developed by the Institute for Hydraulic and Water Resources Engineering, Section for Engineering Hydrology and Water Management (ihwb) of Darmstadt University of Technology - Germany, is a software package for river basin management. It allows for the integrated simulation, analysis and optimization of discharge and pollution loads in rural and urban catchments, including processes in the water body, using physically-based hydrologic approaches (figure 3.5). The reservoir module in BlueM solves the continuity equation for multiple inflow and outflow processes. The continuity equation is solved by linearising the process functions between user-defined nodes [Ostrowski, 2010], thus avoiding time-consuming iterations. Processes can be defined as nonlinear functions of reservoir volume or of any other arbitrary system state and can change over time. This makes it possible to model almost any imaginable operating rule [Bach, 2009].

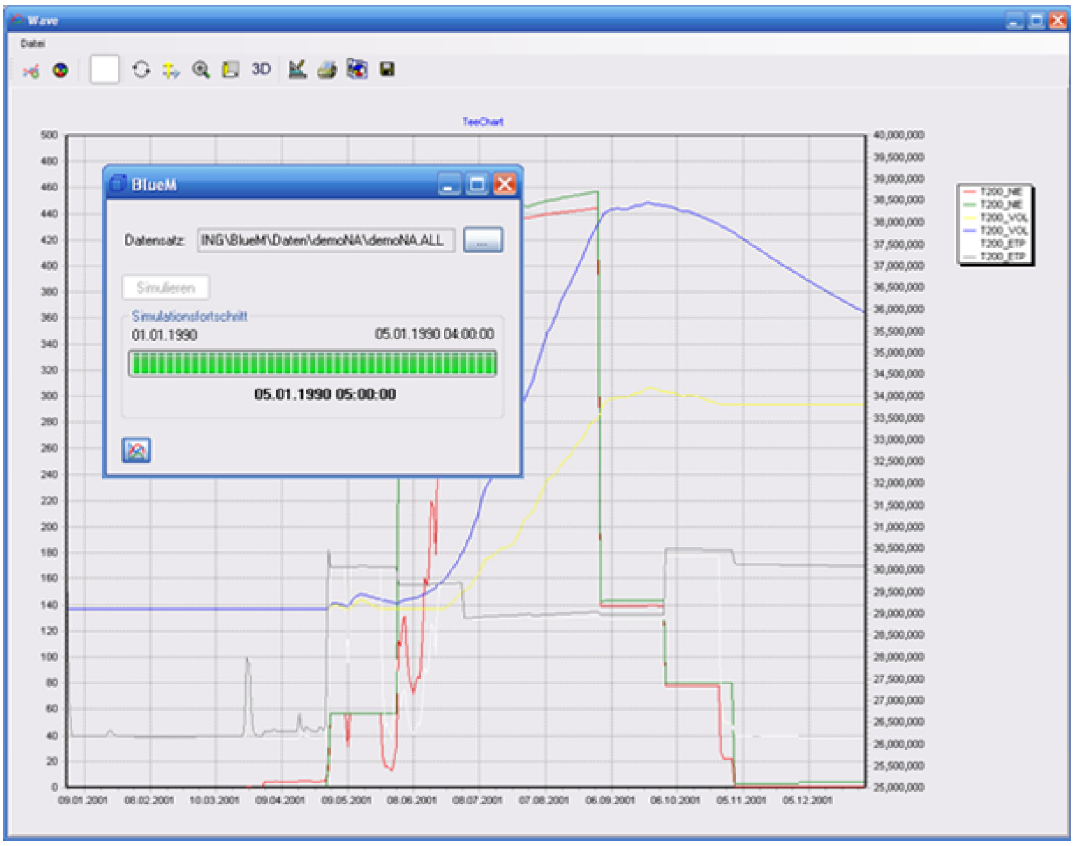


Figure 3.5. Screenshot for the program.

The most obvious advantage of this program surely is its modular structure, which makes it possible for the user to model nearly every kind of reservoir system or river basin. The combination of rainfall-runoff and river-flow-simulation as an input for developing and testing reservoir operating rules and comparing their influence at every point of the system by assigning penalty points to objective functions and monitoring the time series calculated by the model makes it extremely flexible for a large variety of tasks in classical water resource management. This flexibility imposes specific requirements on programming of the user interface as well as simulation modules combined in a modular program system. The modeling framework allows for the evaluation of different climate change scenarios generated by the global climate models, which will be used as input to the model for simulation future inflows to the reservoir.

Figure 3.6 shows the communication between BlueM components and their external usage. The model core BlueM.Sim has two interfaces: an interface which complies with the OpenMI standard and a .NET interface that provides direct access to the model. Additionally, simulation results are also saved in an ASCII file. BlueM.Analyzer is a pure OpenMI-component (implementing the IListener-Interface of OpenMI). BlueM.Wave imports result data from ASCII files. BlueM.Opt can access model engines via a generic interface (implemented as a strategy pattern) or via text files.

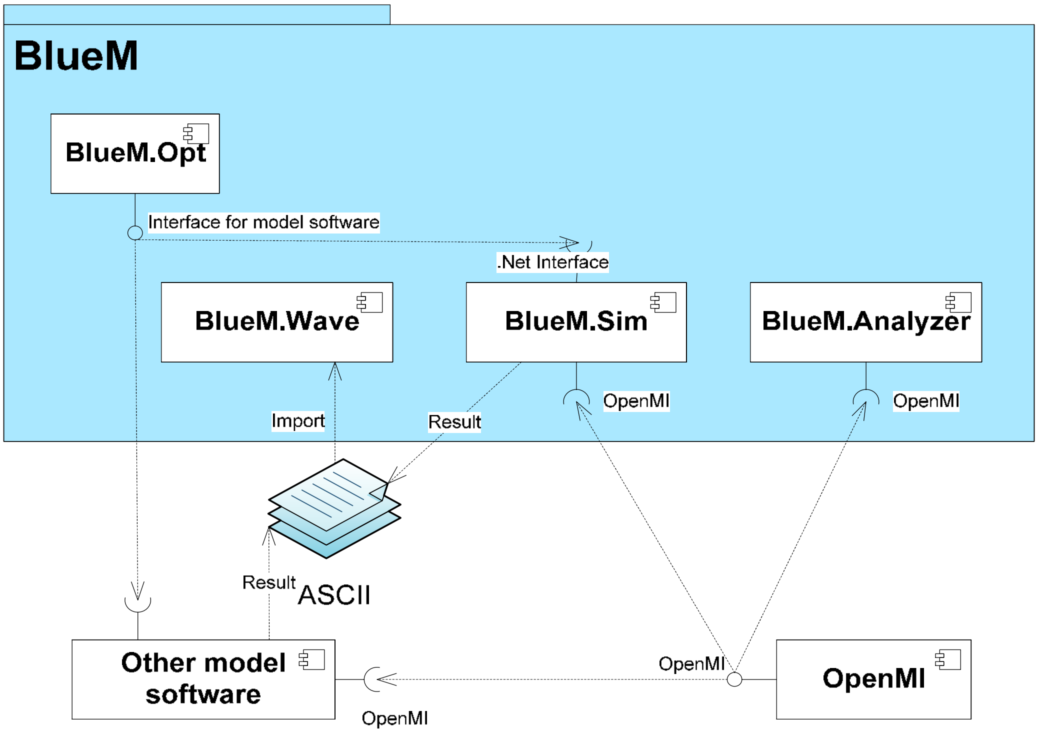


Figure 3.6. Interfaces of the BlueM components and outer word interfaces.

3.4 THE MODELLING APPROACH

The operation of a reservoir is described by the water balance equation under various constraints concerning storage volume, outflow from the reservoir and water losses (see figure 3.7). The water balance equation applied on a monthly basis has the following form:

$$\frac{dV(t)}{dt} = \sum_{i=1}^{n_{in}} Q_{in,i} + \sum_{j=1}^{n_{out}} Q_{out,j} \tag{3.23}$$

$$\frac{dV(t)}{dt} = I_t - Q_t - M_t - D_t - T_t - S_t - E_t \tag{3.24}$$

Where:

$\boldsymbol{I_t}$: Mean inflow to the storage in month t (m^3).
$\boldsymbol{Q_t}$: Amount of water discharged from the storage in month t downstream the dam (m^3).
$\boldsymbol{M_t}$: Amount of water released from the emergency spillway in the dam in month t (m^3).
$\boldsymbol{D_t}$: The water demand for Toshka project (South Valley) in month t (m^3).
$\boldsymbol{T_t}$: Amount of water released from Toshka spillway in month t (m^3).
$\boldsymbol{S_t}$: Seepage losses from the storage reservoir in month t (m^3).
$\boldsymbol{E_t}$: Mean evaporation from the storage reservoir in month t (m^3).
$\boldsymbol{E_t} = ((A_t + A_{t+1}) / 2) * C_t*1000$
$\boldsymbol{A_t}$: Reservoir area at beginning of month t (km^2).
$\boldsymbol{A_{t+1}}$: Reservoir area as at the end of month t (km^2).
$\boldsymbol{C_t}$: Evaporation coefficient pertaining to month t (mm).

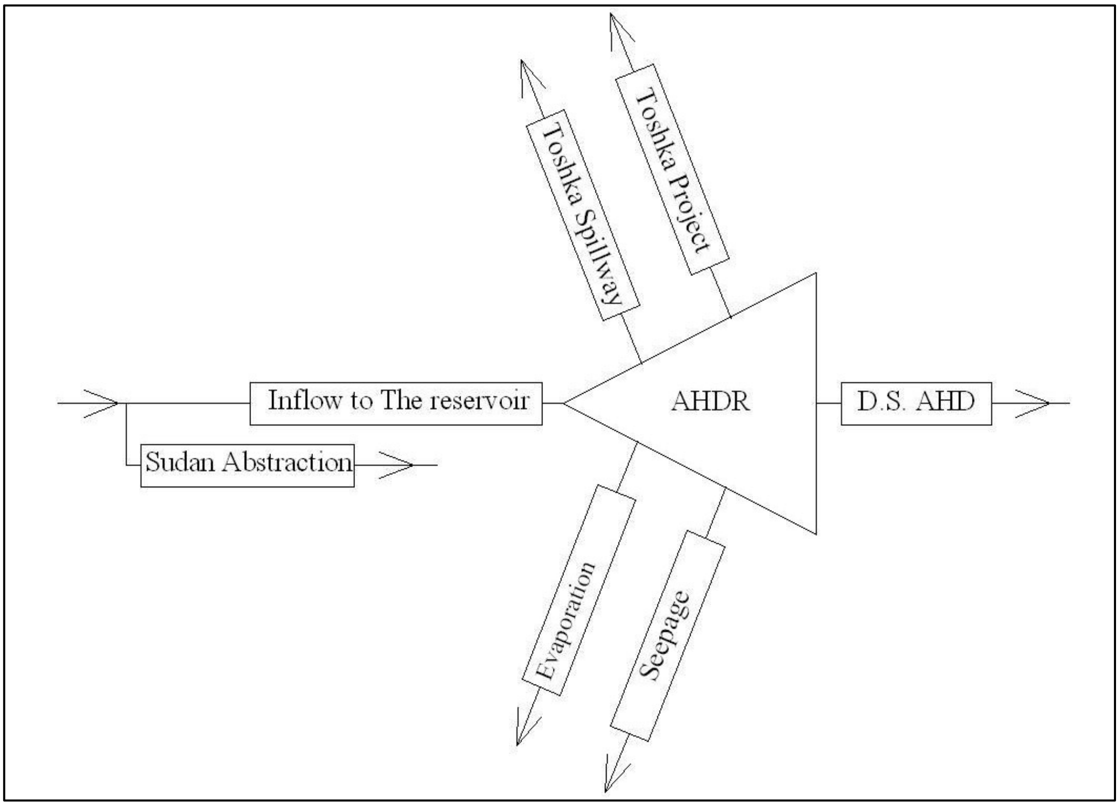


Figure 3.7. Management of the AHDR.

The constraint concerning storage volume V_t is:

$$\mathbf{V_{min} \leq V_t \leq V_{max}}$$

Where: $\boldsymbol{V_{min}}$ = Dead storage volume = 31.60 BCM corresponding to the minimum power pool level (147 m); and

$\boldsymbol{V_{max}}$ = Maximum storage volume = 162.30 BCM corresponding to the maximum power pool level (182 m).

The mean monthly outflow discharge Q_t during month t must satisfy the constraint:

$$\mathbf{Q_{min} \leq Q_t \leq Q_{max}}$$

Where: $\boldsymbol{Q_{min}}$ = Minimum releases = 925 m^3/s (80 MCM/day); and

$\boldsymbol{Q_{max}}$ = Maximum releases = 2890 m^3/s (250 MCM/day).

The model also includes an equation which computes the potential monthly hydropower production as a function of three factors, (1) the volume of water discharged, (2) the gross head of this water, and (3) the efficiency of the couple turbine generator, which varies the amount of power produced. The following functional form represents this relationship:

$$P_t = 9.81 * H_t * Q_t * C_e \qquad (3.25)$$

$$E_t = P_t * K_t \qquad (3.26)$$

Where:

- P_t : Power generated in month t (kW).
- E_t : Energy generated in month t (kWh).
- Q_t : Amount of water turbined for energy generation in month t (m^3)
- H_t : Average height of water above turbines in month t (m) (reservoir monthly mean water level - 110)
  110 meters assumed to be the constant level downstream of AHD (The water level downstream AHD ranges between 107.5 and 113 m above sea levels [Georgakakos et al., 1997]).
- C_e : Efficiency coefficient of turbines and generators (0.85).
- K_t : Number of hours in t month (24* number of hours in month t) (hours).

3.5 THE MODEL CALIBRATION

Calibration of the model was done by using historical data for thirty years (1965-1994) includes characteristics of AHDR, inflow records, downstream releases, spills, evaporation and seepage losses (details of data in chapter 4). Figure 3.8 shows the comparison between results of the computed reservoir elevations and the observed reservoir elevations.

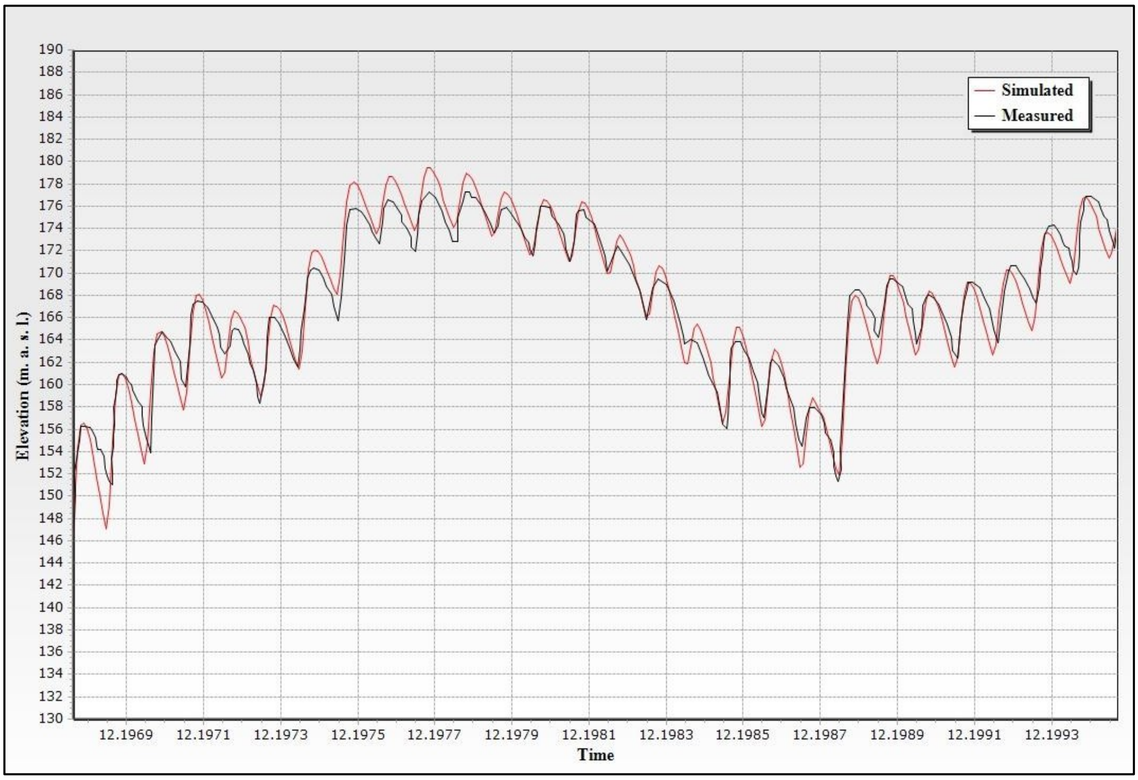


Figure 3.8. Observed and simulated elevations.

4 SYSTEM DESCRIPTION

4.1GENERAL

The main issue of this study is to test whether the AHDR operation strategies, determined for the current conditions, robust to process expected future scenarios. Developing a model to analyses future development of water resources yield and demand and related modifications of the infrastructure for the reservoir is strongly related to the available input data. In order to start a simulation certain input data has to be provided. In this case, the necessary information consists of the following components:

- Characteristics of the AHDR:
  - Reservoir storage capacity.
  - (Elevation-Volume)- (Elevation-Area) curves.
  - Effects of the sedimentation on the storage capacity**.**
  - Reservoir operation policy.
  - Flood Control.
  - Hydropower production from the AHD.

- Inflow Records.
- Downstream Releases.
- Sudan Abstraction.
- Toshka Spillway.
- Toshka Project (South Valley).
- Losses:
  - Evaporation.
  - Seepage.

For each of these components the existing data has to be identified and prepared, which will be described in this chapter.

4.2CHARACTERISTICS OF THE AHDR

4.2.1 Reservoir Storage Capacity

The operation policy of the reservoir is based on dividing the reservoir storage into three zones, illustrated in fig. 4.1. The dead storage zone, that receives sediments during the flood period, has a top elevation of 147 m with total volume of about 31.60 BCM. This zone releases no flow, regardless of the downstream requirements. The live storage zone, which amounts to 89.70 BCM includes the buffer zone and the conservation zone. The buffer zone lies between elevation 147 and 150 m while the conservation zone lies between 150 and 175 m. An additional storage volume of 41 BCM is available for high flood waters. It is between elevation of 175 and 182 m, and brings the total reservoir volume up to 162.30 BCM [Whittington and Guariso, 1983], this amount is increased by 7 BCM in case of raising the water level up to (183m) [NBCBN, 2005].

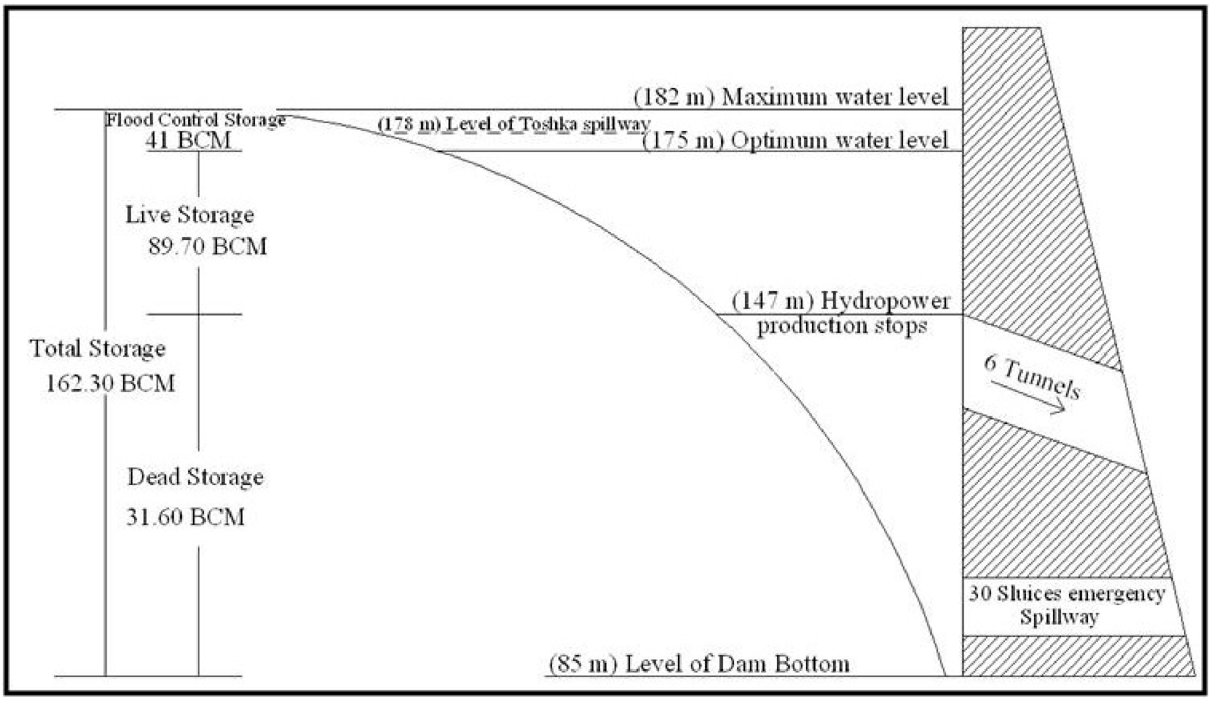


Figure 4.1. Design criteria for the AHD [Source: Whittington and Guariso, 1983; Ragab and Prudhomme, 2002; Dumont, 2009].

Within the live storage zone, the dam operators make their releases to meet downstream requirements, although the total annual release should not normally exceed Egypt's agreed quota (55.5 BCM) [Fahmy, 2001; Dumont, 2009].

4.2.2 (Elevation-Volume)- (Elevation-Area) Curves

The area-elevation-volume relationships are the key to any simulation process, details of storage volume and storage area against elevation for AHDR as provided by Ministry of Water Resources and Irrigation (MWRI) are given in figure 4.2.

The water level at the AHDR still fluctuates from year to year. Figure 4.3 shows the storage level of the AHDR from 1968 to 2007 and corresponding reservoir content over years.

Figure 4.4 shows maximum and minimum values of water level in the AHDR since beginning of the reservoir formation in 1964. In 1978, the AHDR reached its first peak of 178 m.a.s.l., but by 1988 the level dropped to 154 m.a.s.l. Since that time, however, the reservoir has continued to rise again, by 1998 it reached nearly182 m.a.s.l. and remained high all through 2000, even in summer it was much higher than in previous years [Irina et al., 2001].

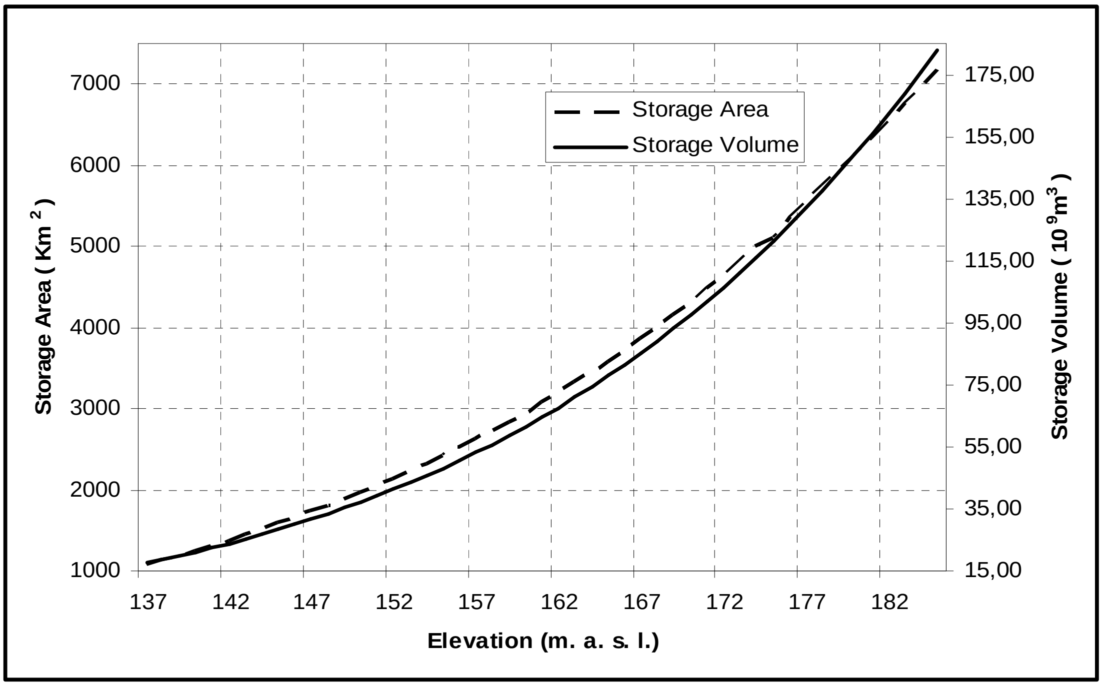


Figure 4.2. (Elevation - Volume – Area) curve [Source: Whittington and Guariso, 1983].

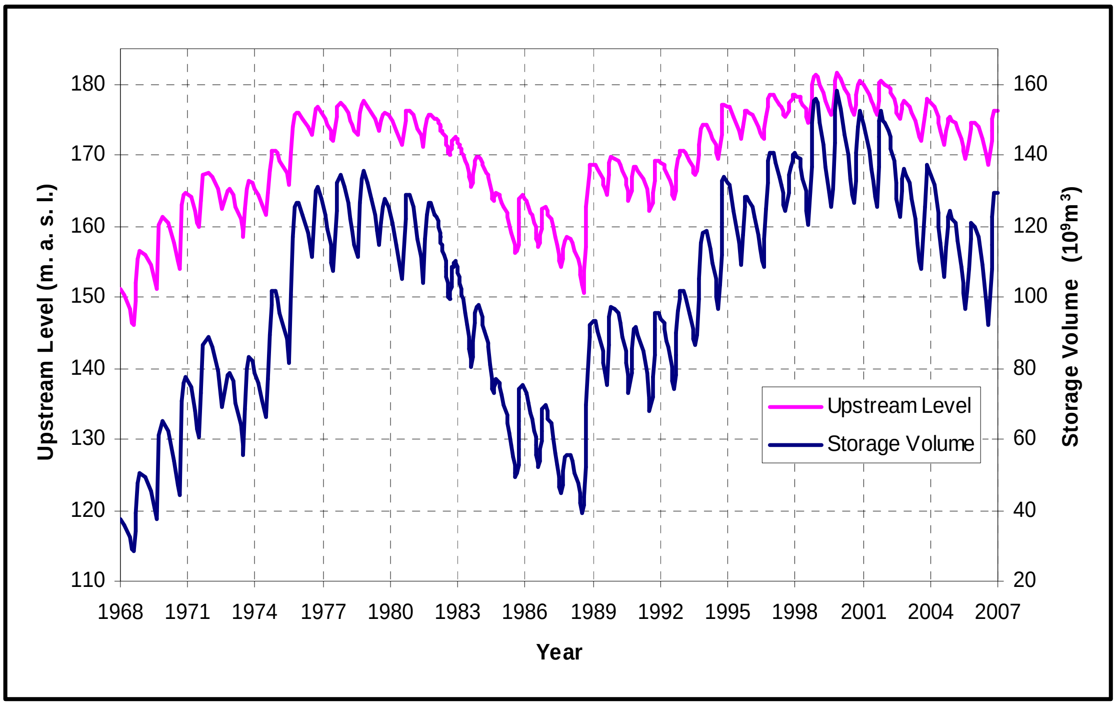


Figure 4.3. Upstream level and storage volume of the AHDR from 1968 to 2007 [Source: NBCBN, 2005; Rayan et al., 2008].

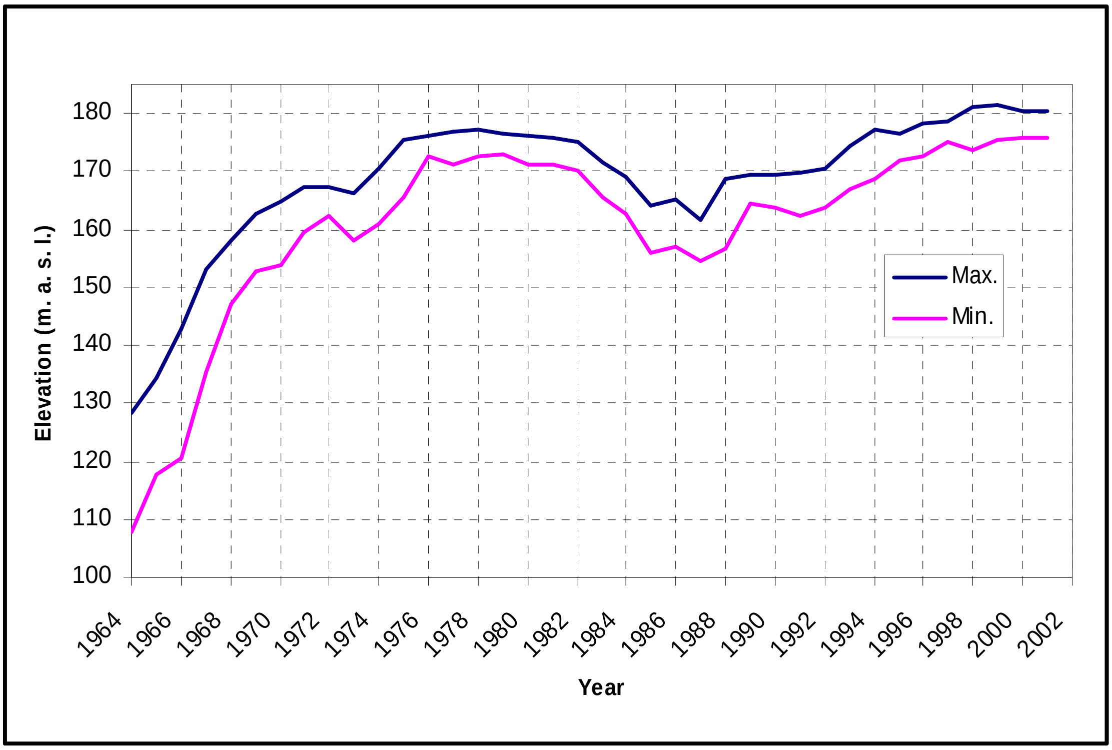


Figure 4.4. Max. & Min. values of water level in the AHDR [Source: Irina et al., 2001].

4.2.3 Effects of Sedimentation on The storage Capacity

4.2.3.1 Sediment deposition in the AHDR

All reservoirs formed by dams on natural water courses are subject to some degree of sediment inflow and deposition. A reservoir changes the hydraulics of flow by forcing the energy gradient to approach zero. This results in a loss of transport capacity with the resulting deposition. The smaller the particles, the farther they will move into the reservoir before depositing. Some may even pass the dam. Deep reservoirs are not fully mixed and are conducive to the formation of density currents [U.S. Army Corps of Engineers, 1997].

The Nile River receives most of its sediment load from the Atbara and Blue Nile rivers, which carry eroded sediments north from the Ethiopian mountains during the seasonal flood period between August and October.

The long-term average (1929-1959) of sediment load that enters the Old Aswan Reservoir at Wadi Halfa (a town in the northern Sudanese state on the shores of the Sudanese section of Lake Nasser) was estimated to be 134 x 10^6 metric tons [NBCBN, 2005]. Prior to the construction and operation of the AHD, in 1964, 9-10 x 10^6 metric tons of suspended sediment were deposited annually in the flood plain of the Nile, while about 93% of the total average annual suspended load of 124 x 10^6 metric tons was carried out into the Mediterranean Sea.

Since the full operation of the AHD in 1968, the flood discharge of the Nile, below the dam, has been greatly modified and more than 98% of the total suspended load has been retained within the reservoir [Shalash, 1982].

Based on a period of record of over 30 years taken between 1929 and 1964, Shalash (1982) established the relationship between the "period" discharge, Q_f, of the Nile during the flood season (August to October), and the "period" suspended solids (SS) discharge, Q_s, at the Kajnarty station, 399 km upstream from the AHD.

The "period" load represents approximately 96% of the total annual SS load. Shalash found that the relation between Q_f and Q_s could be represented by the following equation:

$$Q_s = 0.328 Q_f^{\ 1.49} \qquad (4.1)$$

Where, Q_f is in km^3 and Q_s is in 10^6 metric tons.

Sediment distribution in the AHDR is investigated regularly along fixed 21 cross-sections (figure 4.5) Extensive bathymetric survey is conducted along the fixed cross- sections resulting clear profiles of the reservoirs. Since 1973, investigations and analysis for sediment deposition both upstream and downstream the AHDR has been conducted.

Sediment investigations are carried out three times a year, before, during and after the flood period. The measurements cover a distance of about 220 km (figure 4.5) at the tail zone of the backwater curve, behind which no sedimentation is observed. In this reach fixed measurement stations were selected. The types of records are velocity measurements, suspended sediment sampling, hydrographical survey, freshly deposited sedimentation samples, and chemical analysis of water samples [NBCBN, 2005].

The total amount of deposited sediment was evaluated by assuming gradual distribution to the amount of sediment between each two successive cross sections as shown in figure 4.6. From a comparison of the year 2003 reservoir volume with the year 1964 (original volume), It was estimated that more than 5.2 Milliards tons of sediment deposit in the reservoir [El-Sersawy, 2005]. In addition, the deposition thickness for each cross section of the reservoir was obtained from year 1964, to year 2003 as shown in Table 4.1 and figure 4.7 shows the longitudinal profile along the AHDR.

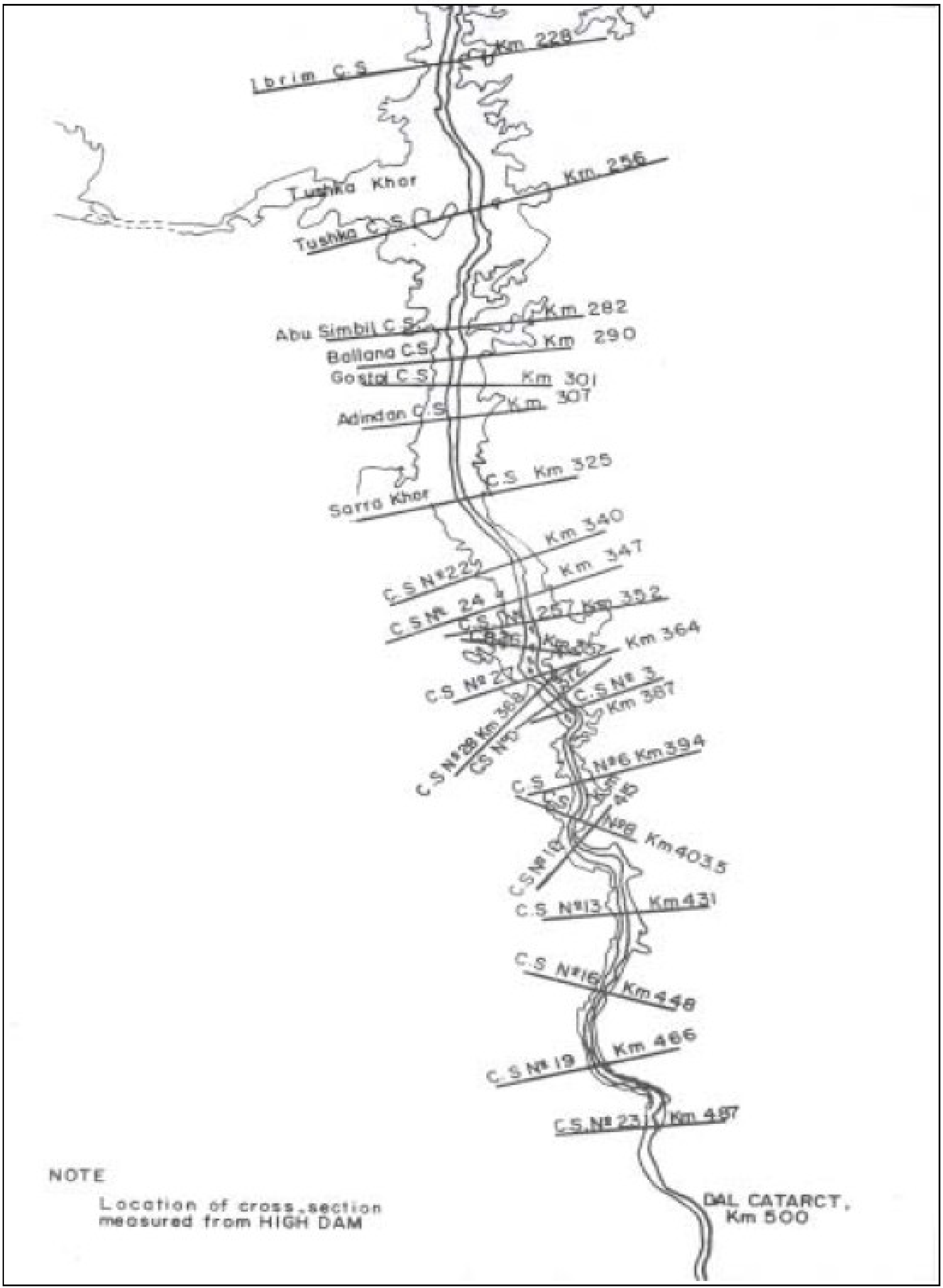


Figure 4.5. Location of cross sections along the AHDR [Source: NBCBN, 2005].

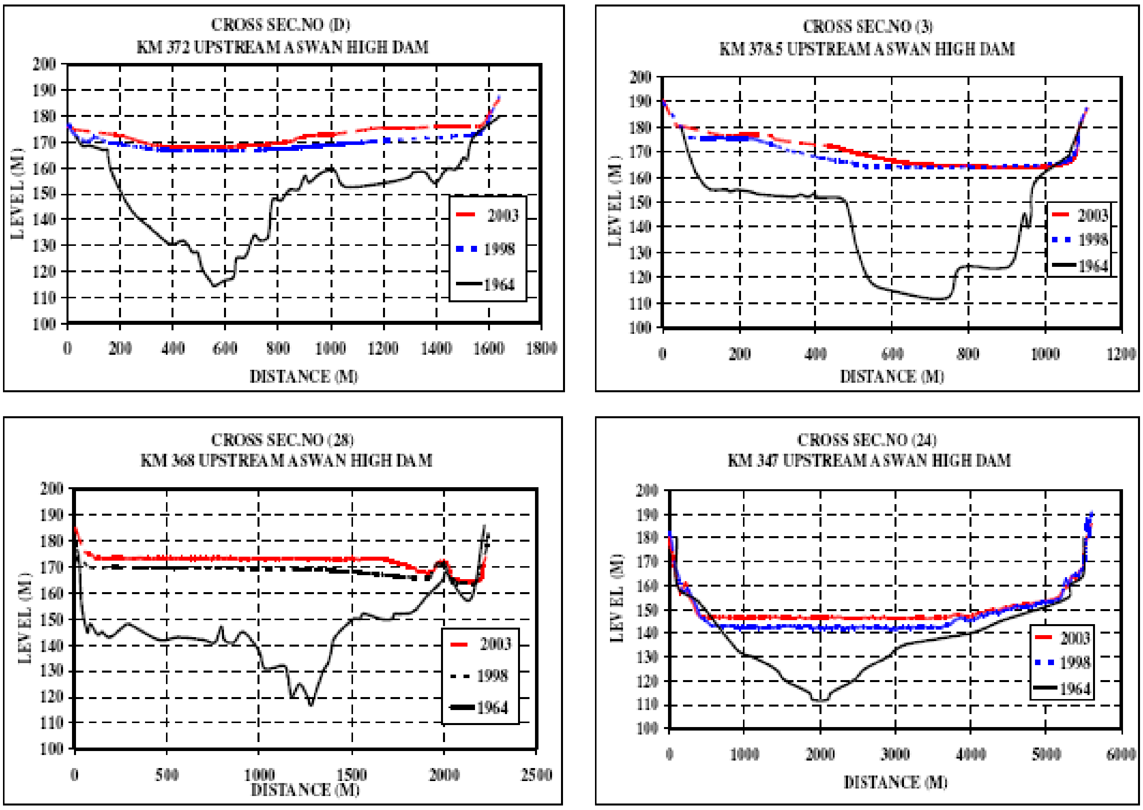


Figure 4.6. Cross sections of the AHDR for years 1964, 1998, and 2003 [Source: El-Sersawy, 2005].

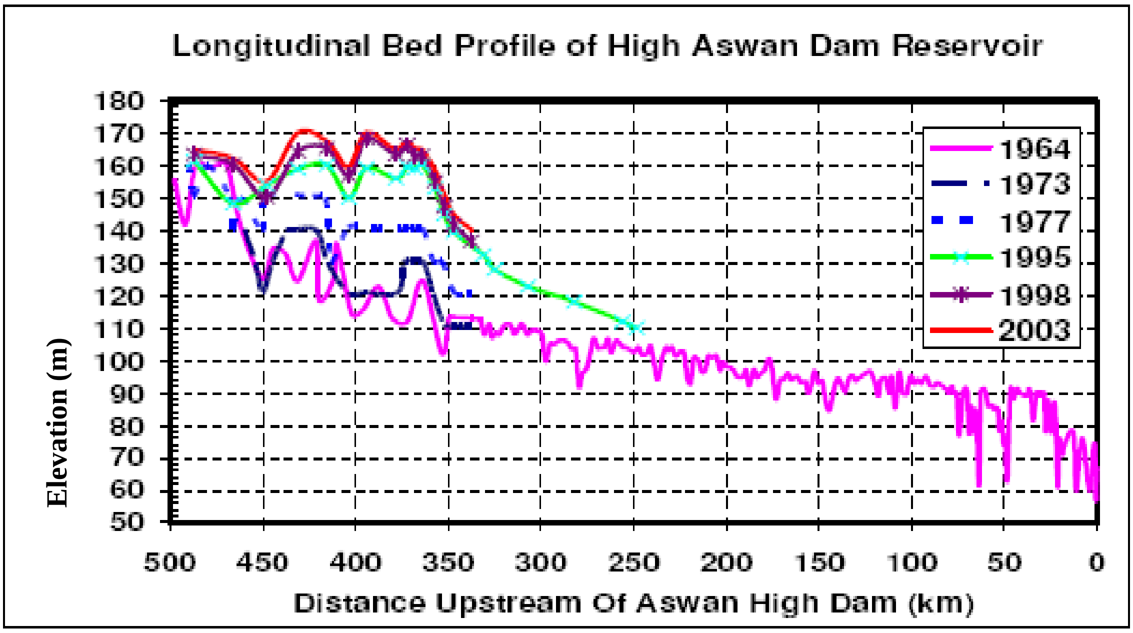


Figure 4.7. Longitudinal section of the AHDR [Source: El-Sersawy, 2005].

No.	Cross Section Name	Cross Section Number	Distance Upstream AHD (km)	Thickness of Sediment Deposition (1964-2003) (m)
1	Daka	23	487	9.52
2	Akma	19	466	15.29
3	Malek Naser	16	448	27.38
4	Dewashiate	13	431	31.59
5	Ateri	10	415.5	35.26
6	Semna	8	403.5	43.52
7	Kagnarty	6	394	59.48
8	Morshed	3	378.5	53.93
9	Gomi	D	372	61.13
10	Madik Amka	28	368	41.38
11	Amka	27	364	27.65
12	Gandal Sanii	26	357	48.66
13	Abdel Khader	25	352	43.36
14	Dogheem	24	347	33.37
15	Dabrossa	22	337.5	29.88

Table 4.1. Names and locations of the hydrographic survey stations in the AHDR [Source: El-Sersawy, 2005].

4.2.3.2 The life span for the AHDR

The additional time before filling estimated in the revision is significant for decision rules governing operation of the AHDR. A number of estimates of the potential life span of the reservoir have been made in the past several years. Published estimates range from as little as 20 years (Sterling 1970) to over 1500 years (Makary 1982). The wide range of differing values is a function of many variables, including computation method, input data, and theoretical assumptions underlying the mathematical approach taken. Such a broad range of values is typical of sedimentation studies performed for areas that lack an adequate historic data base.
Shalash & Makary (1986) indicated that, the mean annual suspended load is about ($130 \times 10^6 \pm 5$) ton, corresponding to a volume of 91.7×10^6 m^3. With a trap efficiency of 98%, the life age becomes 350 years only.
Smith (1990) concluded that, the time forecasted for filling of the reservoir by taking into account changes in the hydrological regime of the Nile after 1964 and another compaction factor, an estimate of 535 years.
Makary (1992) reported that, the total inflow to the reservoir in the period 1964-1989 was 1864 BCM carrying 2861×10^6 ton of sediments of which 2800×10^6 ton have accumulated in the reservoir. The density and the volume of the deposited sediments are 1.34 t/m^3 and 2.09 km^3, respectively. This huge amount has led to the rise of the original bed level. Assuming that, this average rate of sedimentation will continue in future, the estimate of the life age must be about 390 years.

Abdel-Aziz & De Smedt (1992) found that, for the period 1964-1988 the sediment mass, density and volume were 3330 x 10^6 ton, 1.12 t/m^3 and 2.97 km^3, respectively. Would this be the case, the dead storage capacity will then be filled by sediments in about 265 years, which is nearly 50% of the design life-age of the reservoir [Shahin, 1993].

4.2.3.3 Storage volume losses due to sedimentation

From the previous discussion about the sediment deposition in the AHDR, it can be noticed that the current annual sedimentation rates in the reservoir will not affect reservoir storage for several centuries, the total deposited sediment volume was 2477 million m^3 in the time interval 1964-1995. However, compared to the other reservoirs the AHDR has experienced a unique sediment process with rather varying consequences.
It can be noticed that sediment deposition started at the tail zone of the reservoir and steadily progresses northward along the river bed. It seems that sediment deposition will approach the dam after a pretty long period of time. The dead storage capacity (31.60 BCM) was estimated to be sufficient for accumulation of suspended matter over 300-500 years (100-60 million m^3/year, or 150-90 million tons/year) [NBCBN, 2005].

Although it is very early to make such estimates of the unreduced storage capacity of the AHD, the results provide encouraging support to the design estimates. In addition, modification of the reservoir design by construction of the Toshka flood diversion should serve to increase the quantity of suspended load which is carried through the reservoir and thereby increase the overall life-span of the reservoir, considerably [Shalash, 1982].

4.2.4 The Reservoir Operation Policy

The major task of the operation rules is to provide sufficient water supplies and to avoid river damages in addition to maintaining the dam structural safety. The reservoir operation policies were determined by the Ministry of Water Resources and irrigation according to different restrictions such as:

1- Maximum allowed water outflow should not exceed 2890 m^3/sec (250 MCM/day) to avoid excessive erosion and banks overtopping.
2- The water levels upstream High Aswan Dam should be kept at 175.00 m at the beginning of water year (August 1st) to fulfill high and low flood requirements, Raising upstream water level at the beginning of the water year may have some positive effects due to the addition water storage availability for next low floods years. However, raising water level may cause some side effects such as increasing water losses, higher risks for future high floods (dam safety risk and higher water discharges damaging).
3- The minimum allowed water discharges should be released to fulfill irrigation, navigation, drinking and other requirements and Sudan abstractions [Sadek and Aziz, 2005].

4.2.5 Flood Control

When the water level upstream the dam reaches an elevation between 178 and 183 m, the surplus water will be released, if necessary, by means of 30 sluices emergency spillway, on the western bank of the Nile, to allow the passage of 5000 m^3/sec of water. This water pours back into the Nile downstream of the dam. The selected future scenarios in this study were based on releasing Spillway discharges (if required) only above the level of 180.00 for the reservoir. Figure 4.8 shows the discharge curve for the AHD emergency spillway.

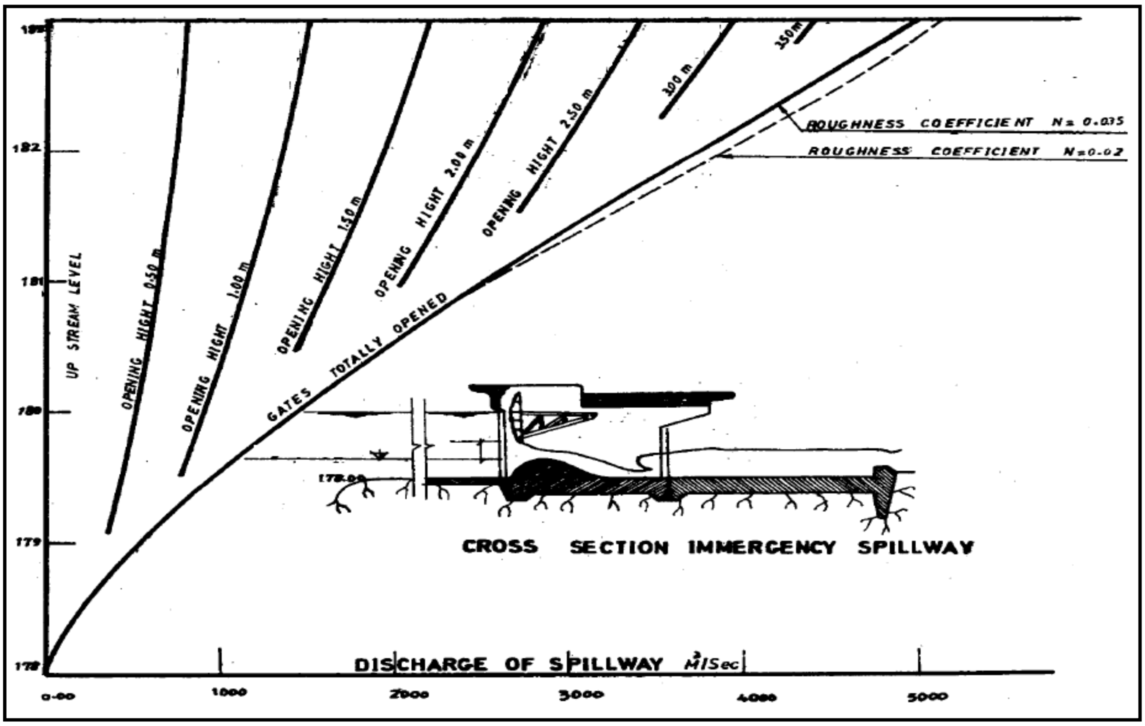


Figure 4.8. Discharge curve for the AHD emergency spillway [Source: Saad, 2008].

4.2.6 Hydropower Production from the AHD

The power station of the AHD comprises 12 generating units with a capacity of 175,000 KW each. Each generating unit is equipped with a Francis turbine [Moussa et al., 2001].

The first unit was put into operation on 15 October 1967, the AHD hydropower plant began generating power at that time with an output of 71 GWh in 1967, and gradually increased production to about 3700 GWh in 1972 against a total power generation in Egypt of 7400 GWh, i.e. about 50% of total power generated at that time. Production by the AHD hydropower plant is now about 8000 GWh/year [Abu-Zeid and El-Shibini, 1997].

Respectively, the power generated from the turbines is transmitted through transmission overhead lines to the load centers on voltage Levels 500, 132 kv [MEE, 2005]. The turbines are designed to operate within 50 to 74 meters net hydraulic head, implying that power generation below 160 meters wears out the turbines and therefore not desirable [Georgakakos et al., 1997]. Information about existing generated electricity from the AHD as provided by the Ministry of Electricity and Energy (MEE) is presented in table 4.2.

Item	2004/2005
Discharge (m^3/s)	1805.87
Average head (m)	65.11
Upstream water level at end of the year (m.a.s.l.)	170.12
Downstream water level at end of the year (m.a.s.l.)	109.04
Efficiency (%)	89.5
Max. load (MW)	1980
Generated energy (GWh)	9049
Max. generated energy (GWh/d)	40
Min. generated energy (GWh/d)	10.10

Table 4.2. Hydropower Data (2004-2005) [Source: MEE, 2005].

The water discharge gradually rises from 1504.63 to 2546.30 m^3/s (130 to 220 MCM/day) from April until August and then declines to 1388.89 – 1504.63 m^3/s (120 - 130 MCM/day) in November. The minimum flow period is November to March. The minimum discharge reaches about 1250 m^3/sec (108 MCM/day). The central load dispatching centre schedules generation for successive 10 day periods based on projections of water flow received from the Ministry of Water Resources and Irrigation. Daily modifications are introduced according to actual conditions. Figures 4.9 and 4.10 illustrate the chronological variations of the total flow, the average discharge rate and the upstream, downstream and the average head for the AHD [Rashad and Ismail, 2000].

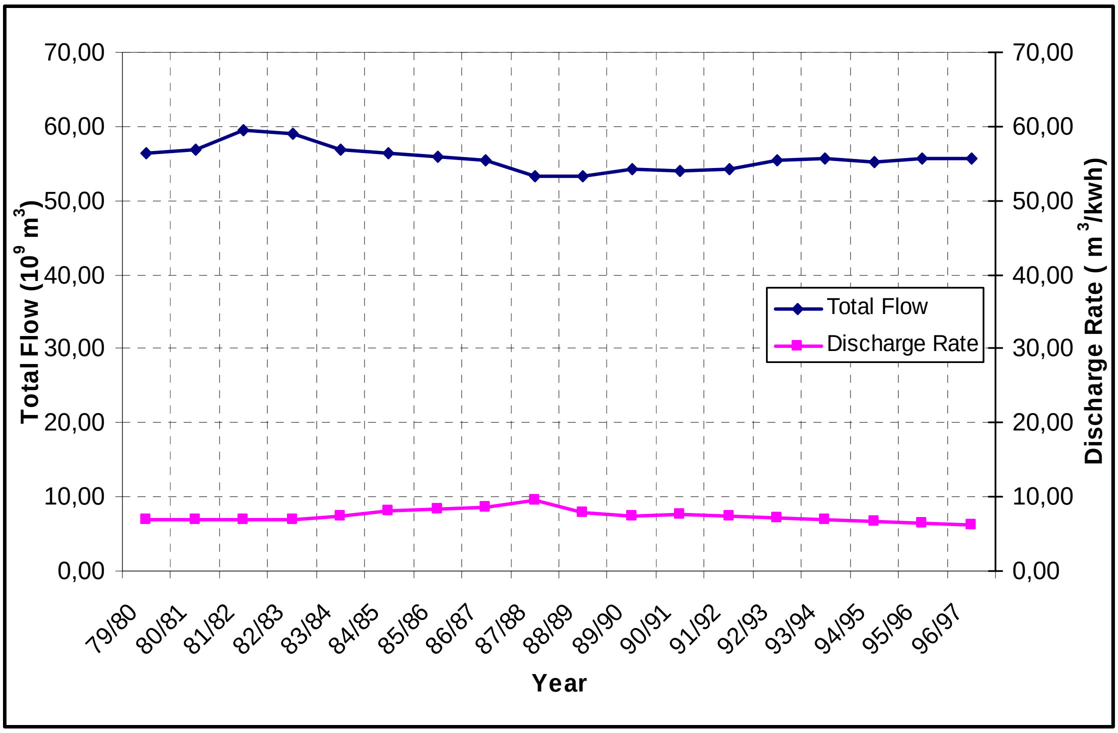


Figure 4.9. Hydro-statistics: total flow and the discharge rate [Source: Rashad & Ismail, 2000].

Figure 4.11 displays Analysis of Egyptian High Dam energy generation 1979-2005. The trend was obtained by linear regression fitting. The curve "real minus trend" shows that there were periods of lower and higher than expected production of hydropower.

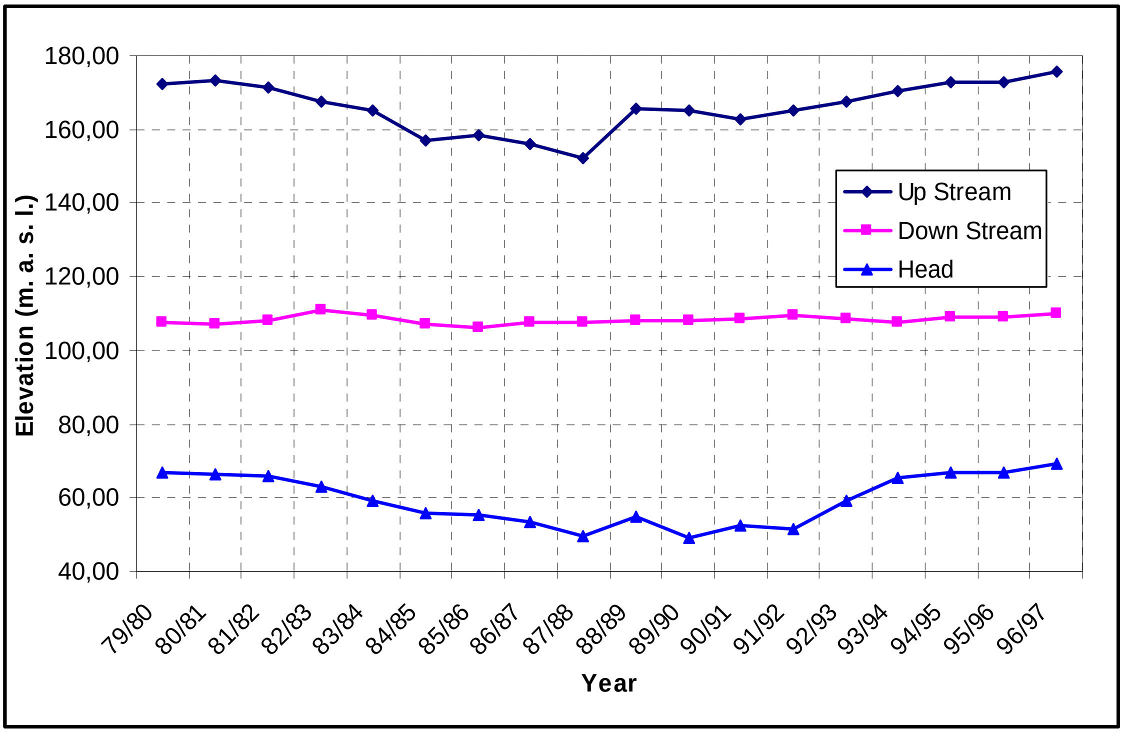


Figure 4.10. The AHD hydro statistics: water level at the end of each year, the upstream, the downstream and average head [Source: Rashad and Ismail, 2000].

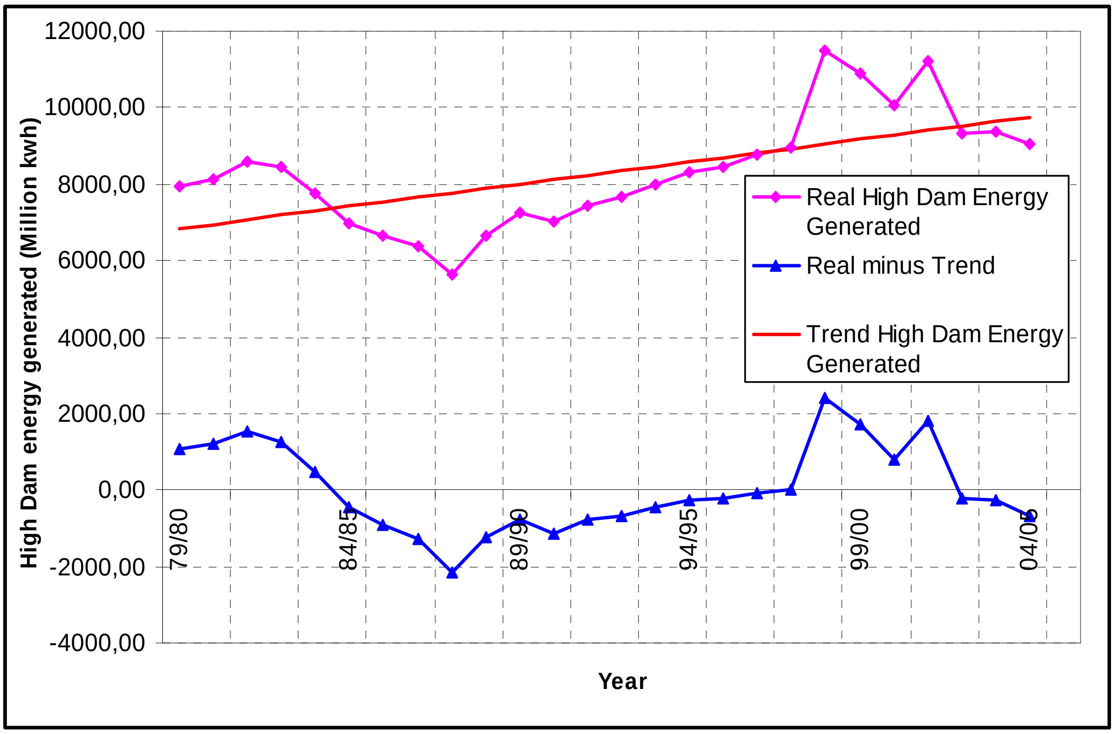


Figure 4.11. Analysis of the AHD energy generation 1979-2005 [Source: Rashad and Ismail, 2000; MEE, 2005].

4.3 ANALYSIS OF INFLOW RECORDS

4.3.1 Analysis of Annual Flows

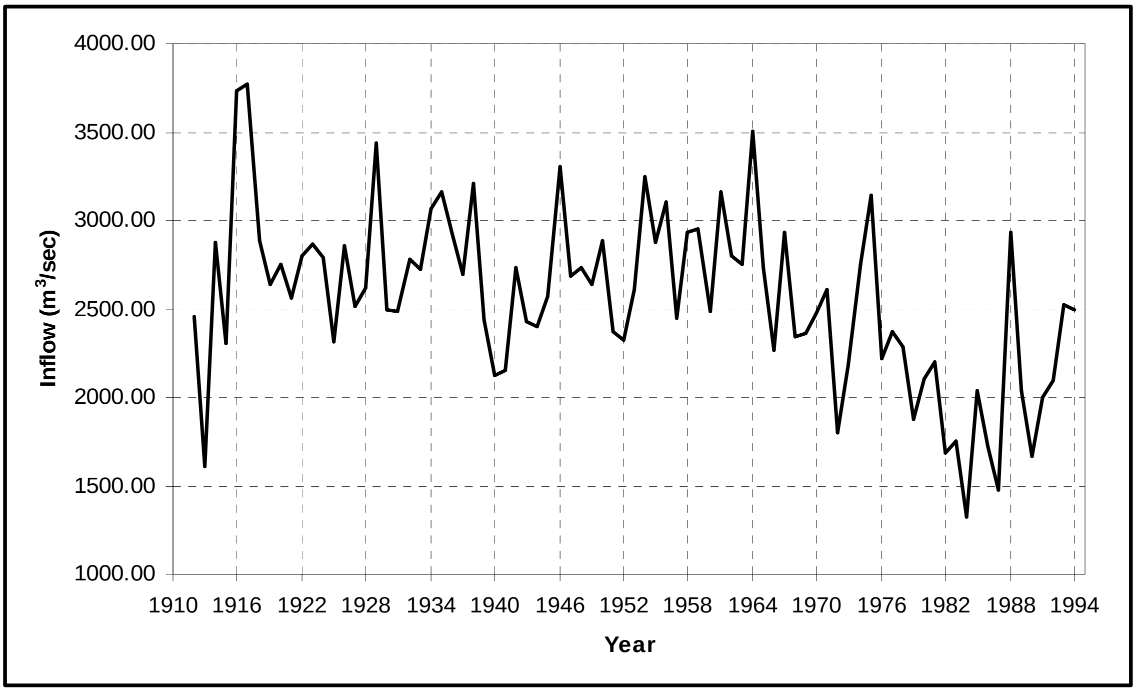


Figure 4.12. The average annual inflow of the historical data at Dongola from (1912-1994) [Source: Ghaas, 1998; Sutcliffe & Parks, 1999].

Annual inflow records at Dongola in Sudan (representing the inflow to the AHDR [Georgakakos & Yao, 1999; Fahmy, 2001; Yao & Georgakakos, 2003]) have been collected and published by Ghaas, 1998 and Sutcliffe & Parks, 1999, in order to get an idea about the high and low flows for 83 years during the period from 1912 to 1994 (figure 4.12).

Table 4.3 shows the statistical analysis of annual inflow data at Dongola. This analysis represents minimum and maximum values, the arithmetic mean, the median (centre value), and standard deviation (STDV). The mean annual flow over the period 1912-1994 was 2259 m^3/s (80.70 BCM) with a standard deviation of 488 m^3/s (15.40 BCM). The mean flow varies significantly depending upon the period considered, the mean annual flow from 1912-1964 (before operation of AHD) was 2754 m^3/s (86.86 BCM) with a standard deviation of 398 m^3/s (12.56 BCM). The mean annual flow from 1965-1994 (after operation of the AHD), on the other hand, was 2214 m^3/s (69.81 BCM) with a standard deviation of 443 m^3/s (13.98).

Statistics	Min	Max	Mean	Median	STDV
1912-1994	1328	3769	2559	2569	488
1912-1964	1606	3769	2754	2731	398
1965-1994	1328	3144	2214	2211	443

Table 4.3. Historical data statistical analysis of annual flow at Dongola during different periods (Flows expressed in m^3/s).

4.3.2 Analysis of Monthly Flows:

The analysis of the annual flows is a very comprehensive description of the Nile hydrology. The characteristic feature of the Nile is rather the variation in the monthly flows. As illustrated in table 4.4, it can be noticed that the hydrological cycle begins with the first rains of July; the flow then increases until September. The period from March to June is substantially drier, a sharp drop corresponding to the dry season occurs in November. Figure 4.13 shows the variation in mean flows at Dongola during different periods.

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec.
	(1912-1994)											
Min	648	495	383	405	328	385	832	2867	2234	1562	908	697
Max	2146	2099	1795	1751	1620	1743	3733	10117	12229	9035	4706	2635
Mean	1261	991	816	900	855	854	1914	6875	7596	4690	2413	1543
Median	1273	948	802	933	828	790	1877	7129	7777	4778	2349	1562
STDV	281	272	227	291	318	284	553	1571	2013	1526	768	379
	(1912-1964)											
Min	784	586	470	405	328	385	832	2867	5169	2934	1597	1116
Max	1997	1798	1795	1751	1620	1581	3733	10117	12229	9035	4706	2635
Mean	1307	992	834	776	702	781	1925	7258	8528	5478	2769	1704
Median	1303	954	832	706	645	706	1866	7317	8603	5525	2758	1695
STDV	244	243	226	256	260	274	576	1479	1430	1174	688	324
	(1965-1994)											
Min	648	495	383	627	631	418	988	3415	2234	1562	908	697
Max	2146	2099	1639	1522	1608	1743	3352	8475	11419	5675	2546	1967
Mean	1179	989	783	1120	1125	984	1893	6198	5951	3298	1784	1259
Median	1109	938	732	1115	1138	962	1916	6365	5795	3061	1724	1210
STDV	326	322	230	207	217	259	519	1520	1844	988	420	298

Table 4.4. Historical data statistical analysis of monthly flow at Dongola during different periods (Flows expressed in m^3/s).

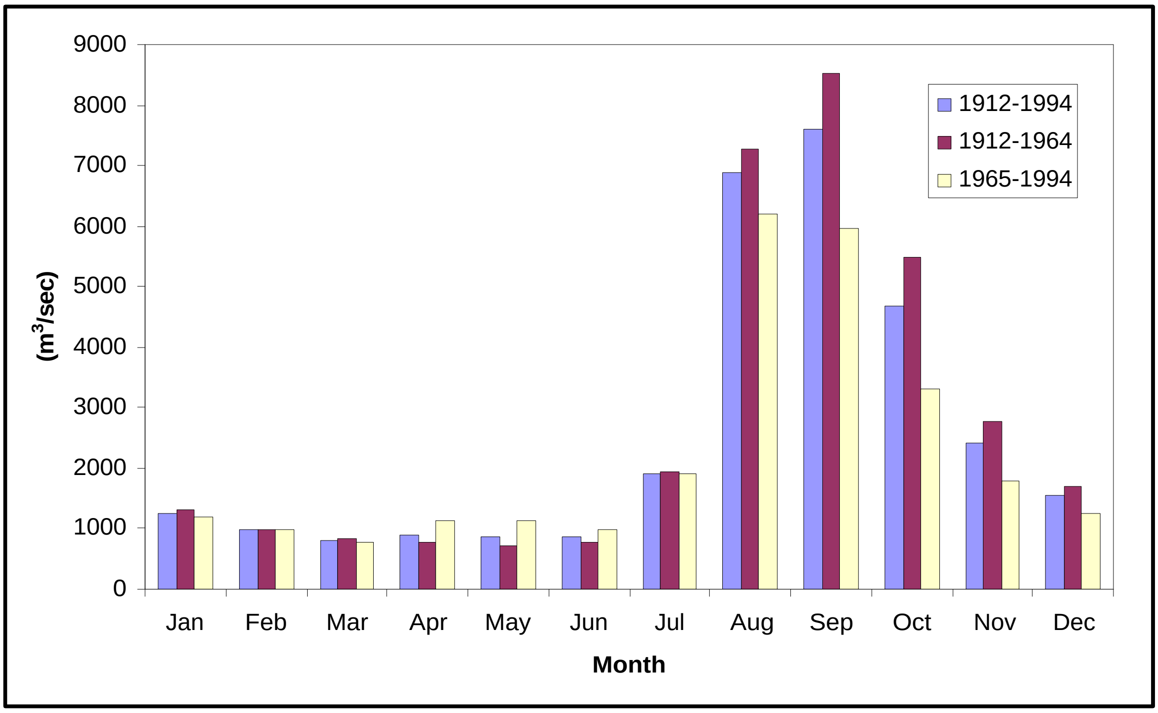


Figure 4.13. The variation in mean flows at Dongola during different periods.

4.4 DOWNSTREAM RELEASES

The release from the AHDR is the annual share of Egypt according to the treaty with Sudan (55.5 BCM) plus possibly the volume released for safety reasons if the water level in the reservoir becomes very high, figure 4.14 shows the annual releases from the dam during (1968-2001).

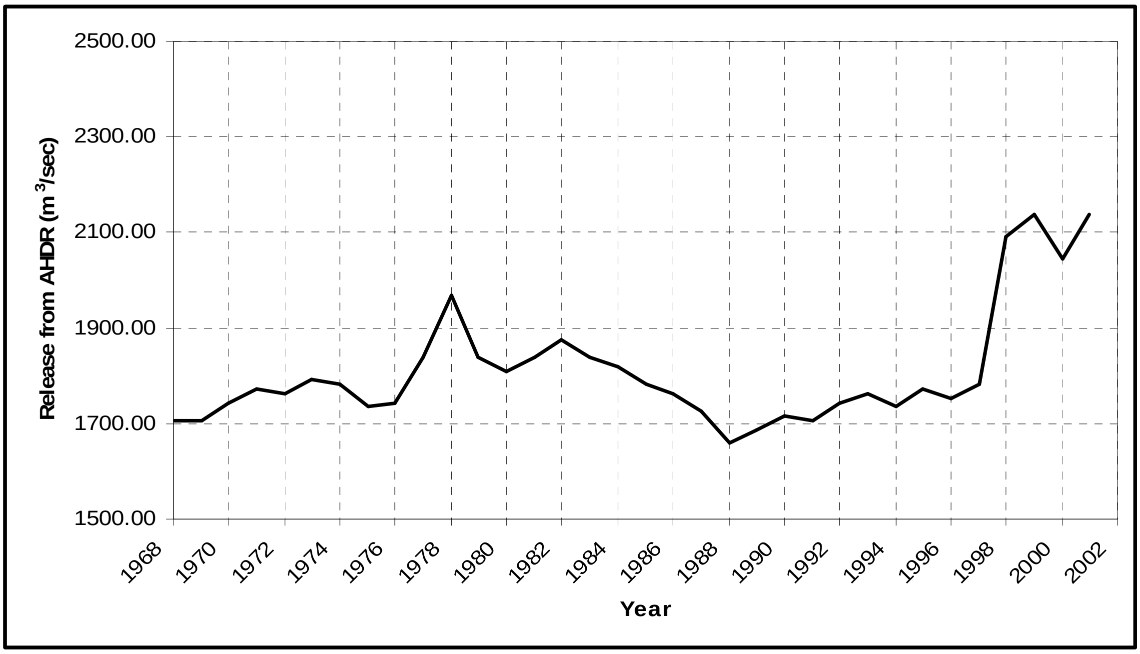


Figure 4.14. Annual releases from the AHD (1968-2001) [Source: Ghaas, 1998; Sutcliffe & Parks, 1999; Delft Hydraulics, 2004].

The monthly outflows (releases) are fixed to meet exactly the predefined downstream water requirements unless the maximum or minimum capacity constraints are violated. These releases from the dam may be reduced according to a reduction factor (sliding scale) if the water levels of the reservoir drop down to the limits of the minimum reservoir level. The same reduction factor is applied to Sudanese abstractions, as well. In case of high flood, where the maximum capacity constraints is violated, the releases may be increased in order to keep the stored water in the reservoir at the maximum level without violating the maximum discharge constrain in the river downstream [Fahmy, 2001].
In this simulation, the monthly releases from the reservoir to Egypt, approximately the 1982 discharge program of Aswan High Dam Authority, are used (table 4.5).

Month	m3/s
January	1347
February	1587
March	1720
April	1701
May	1908
June	2513
July	2581
August	2095
September	1469
October	1422
November	1430
December	1347
Average	1760

Table 4.5. Monthly releases from the AHD [Source: Ghaas, 1998; Sutcliffe & Parks, 1999].

4.5 SUDAN ABSTRACTION

According to an Agreement with Egypt signed in 1959, Sudan’s share of the water available from the Nile is 18.5 BCM/yr. These allocation are based on an average natural inflow into the reservoir of 84 BCM/yr (period 1900-1959) and an estimated 10 BCM/yr of reservoir losses. However, the actual use of Nile water by Sudan during the past period was about 14.5 BCM/yr. Because of this lower abstraction and the higher than average flows in recent years, the level in into the reservoir rapidly rose and significant volumes of were spilled to the Toshka depression through the Toshka spillway [NWRP, 2005].
Figure 4.15 shows the monthly withdrawals to Sudan in the years 1975 and 1980 [Whittington & Guariso, 1983].

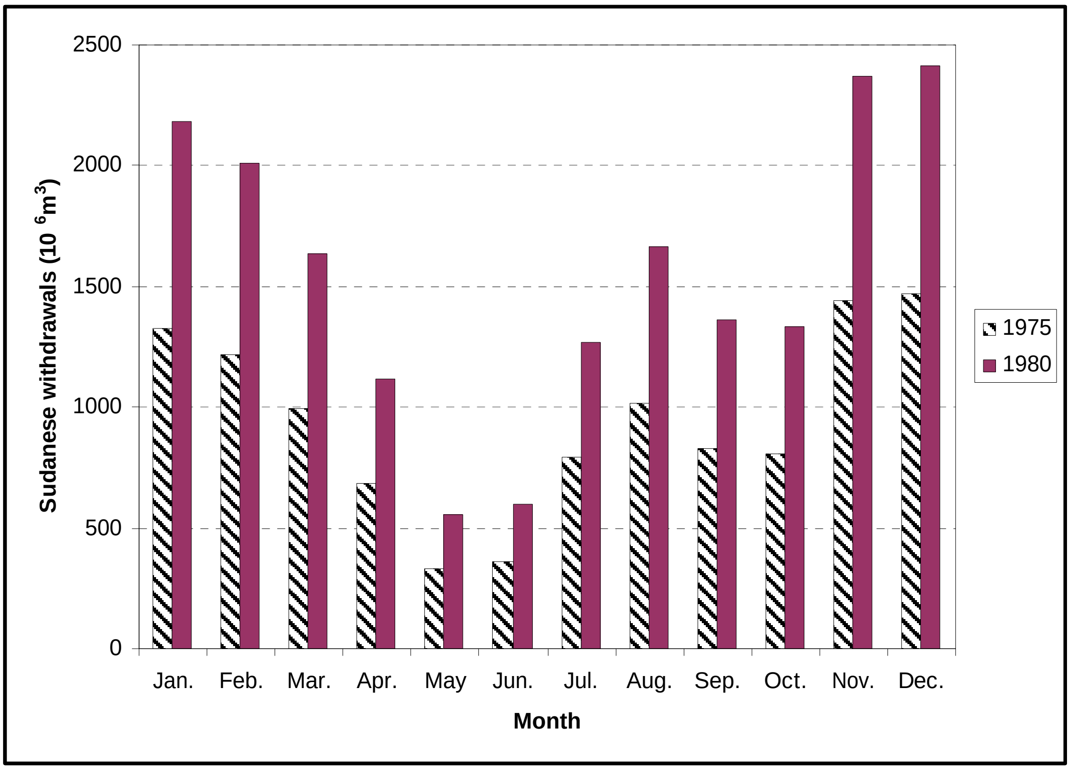


Figure 4.15. Monthly withdrawals to Sudan in the years 1975 and 1980 [Source: Whittington & Guariso, 1983].

4.6 **TOSHKA SPILLWAY**

According to rules of operating the AHDR, the flood control capacity must be emptied down to level 175 m. before the arrival of the following flood, this will result in releasing high discharges, that may reach 4050-4630 m^3/s (350-400 million m^3/day). In this case, further degradation is expected. This may affect the river bed, downstream the control structures existing on the river, the canal intakes and water pumping stations etc. To avoid this it was decided to link lake Nasser at (khor Toshka) to (Toshka depression) in the western desert by an artificial canal to act as additional spillway [Mahmud et al., 2006].

Toshka spillway has played an important part in flood control and management, during years 1998-2002, the flood classification in these years was "high floods". The total discharges passed through Toshka Spillway during this period were 41 BCM [Mahmud et al., 2006].

The project was executed during 1978-1982. It is located 250 km upstream of the Aswan High Dam on the left side of the Nile (figure 4.16). The project includes of Khor Toshka, Toshka canal and Toshka depression.

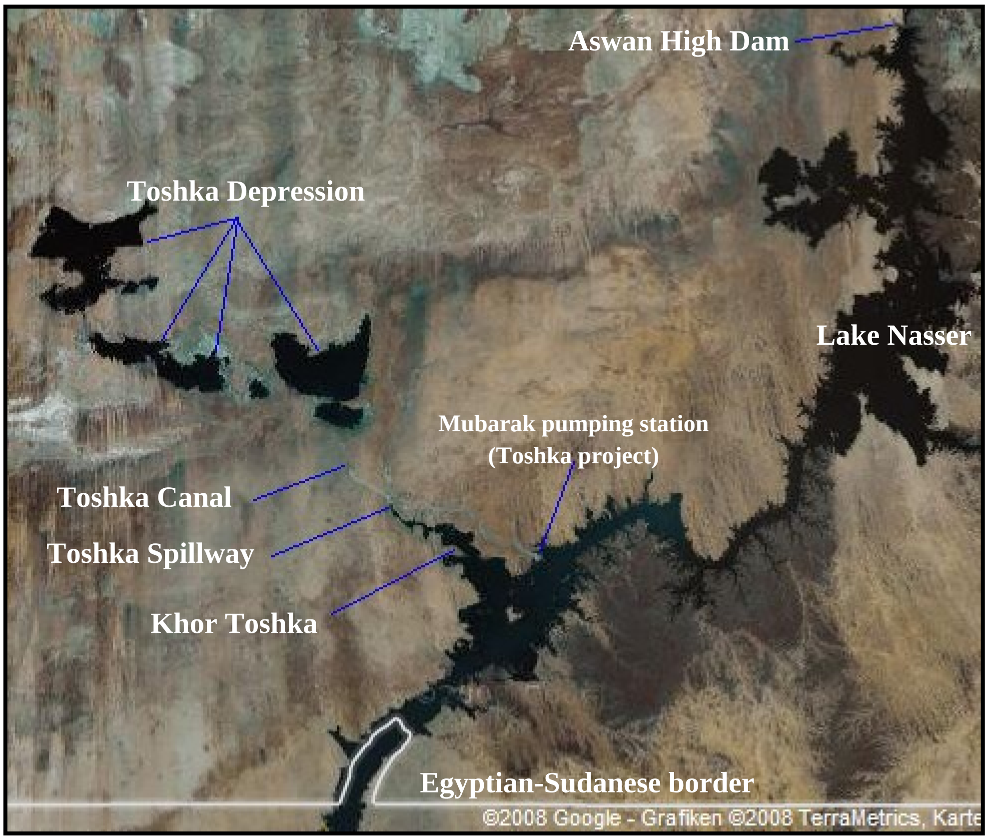


Figure 4.16. Overview of Toshka spillway [Source: Google, 2008].

- Khor Toshka

It is a natural waterway leading to the Nile before the construction of the Dam. It extends in the north west towards Lake Nasser, and it is 56 km long from the centre line of the course of the Nile.

- Toshka Canal

The Toshka spillway canal starts at the end of the Khor Toshka (figure 4.17). The hydraulic parameters of Toshka canal as the following:

- Length = 22 Km.
- Bottom width at the entrance = 750 m.
- Bottom width at the end = 275 m.
- Bottom level at the entrance = 178 m.
- Bed slope = 15 cm/km.
- Side slopes = 2:1
- Max. discharge = 2890 m^3/sec (250 MCM/day).

The spillway consists of a concrete sill with a level of 178 m.a.s.l. The flow passes over the sill when the upstream water level exceeds 178 m.a.s.l. An ogee weir with a crest level of 176 m.a.s.l. is constructed at the end of the Toshka spillway canal at km 20.50 [NBCBN, 2005].

The functions of the weir are to measure the discharge, control the flow, and maintain the stability of the upstream canal. The drop structure has a bed level at its upstream point of 175 m.a.s.l. and at its downstream point of 172 m.a.s.l. [NBCBN, 2005].

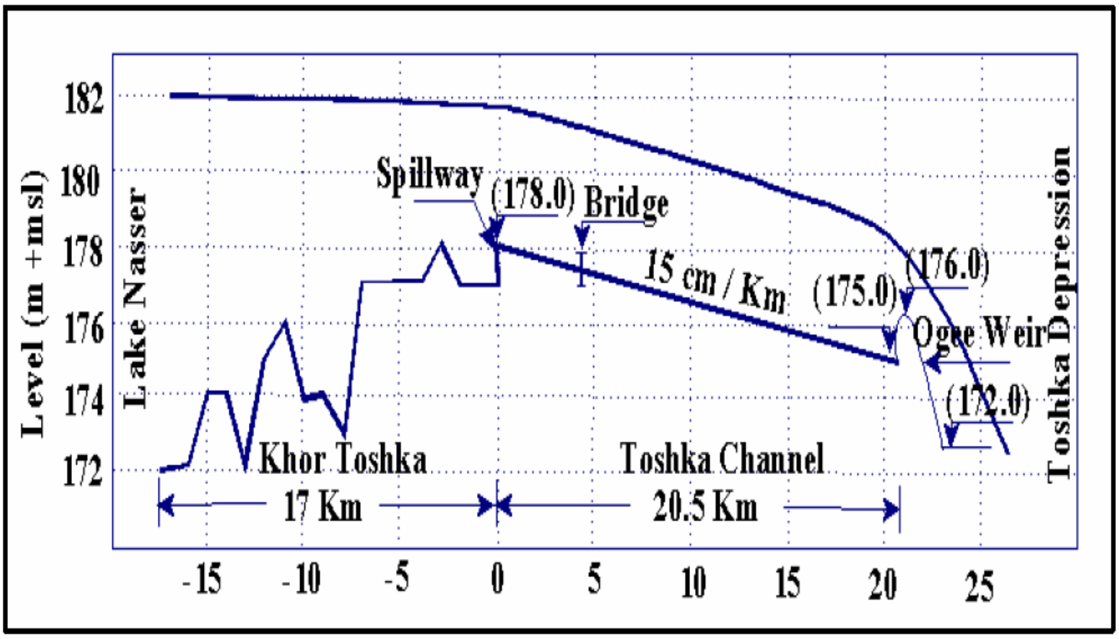


Figure 4.17. Longitudinal sections through Toshka spillway [Source: NBCBN, 2005].

- Toshka depression

It is a natural depression in the western desert (250 km south of the AHD and 56 km west of Lake Nasser) consists of 4 basins between contour (112.00) and (180.00) m.a.s.l. Its surface area is 6000 km^2 at level (180.00) m.a.s.l. where it is surrounded by mountain edges. The total storage of the depression is 120 milliard m^3 at level (169.00) m.a.s.l. [Mahmud et al., 2006].

4.1.1. Toshka Spillway Outflow

In the case that the water level is higher than 178 meters, the excess water is diverted into Toshka depression through the free spillway of Toshka. The discharge is computed using a weir equation form [Fahmy, 2001]:

$$\mathbf{Q = \beta * (H-H_{ot})^{g} * 30.40 * 10^{-3}} \qquad (4.2)$$

Where:

β is 19.0

g is 1.6667.

H is the average water level.

H$_{ot}$ is the crest level of the diversion (178 meters).

Using the measured data of the water level upstream the AHD and the discharge of the Toshka channel for flood of year (1998/1999), another linear regression analysis equation is obtained by Abdel-Moteleb and Saad (2001). The equation is as follows:

$$Q = 519.84\ H - 93028 \qquad (4.3)$$

Where:

Q is the discharge in m^3/s.

H is the water level upstream the AHD in meters.

The calibration for the spillway indicates that the maximum discharge of the channel will be about 137 MCM/day (1586 m^3/s) at a water level of 182 m, instead of 250 MCM/day (1894 m^3/s) which was the designed discharge [Abdel-Moteleb and Saad, 2001]. So, the black line in figure 4.18 was chosen in the simulation to presents the relation between average water level and the discharges to Toshka spillway.

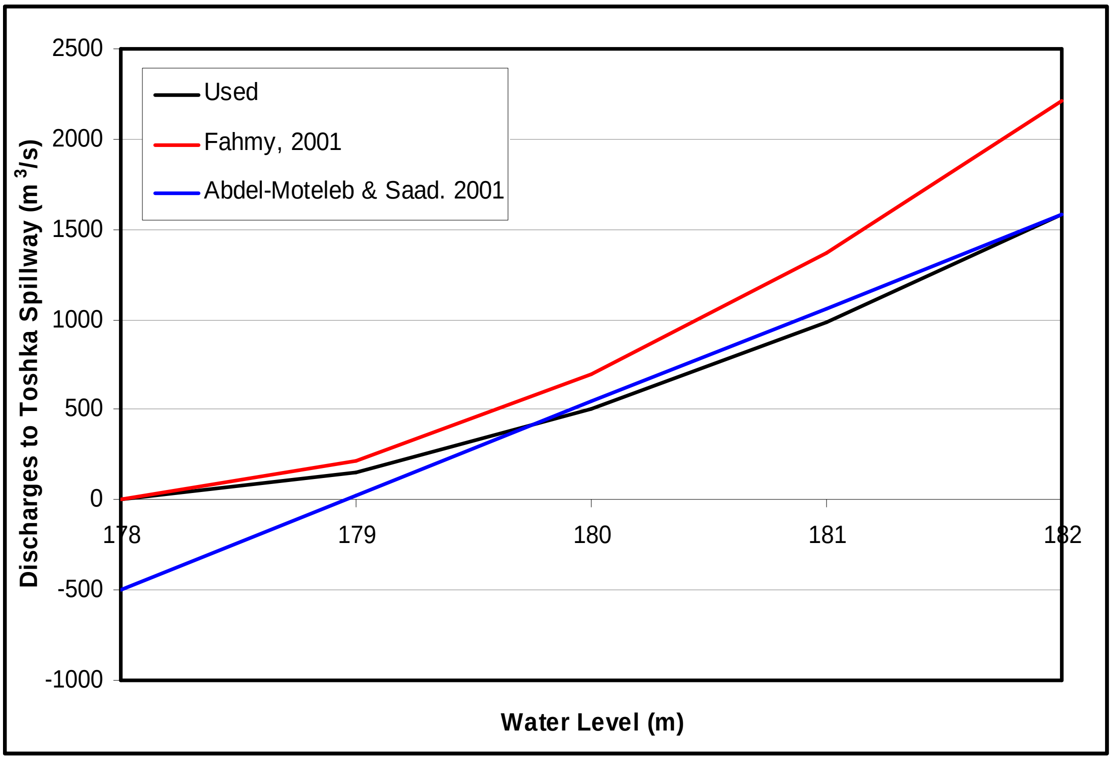


Figure 4.18. The relation between average water level and the discharges to Toshka spillway.

4.7 TOSHKA PROJECT (SOUTH VALLEY)

The Toshka Project was developed in the Western Desert as part of the policy of the Egypt Government to increase the inhabited area of Egypt. The present plan comprises the development of irrigated agriculture on 226,800 hectares near the AHDR (figure 4.19). The water demand (4 to 5 BCM per year) is taken from the AHDR through pumping. In view of the variable levels in the lake (from 147.5 to 178.5 m.a.s.l., the (submerged) inlet has been located at about 140 m.a.s.l., well below the lowest expected level. The water flows through tunnels to the Mubarak pumping station, built in a deep excavated pit. The water is pumped to a level of about 200 m to reach the starting point of the canal. The capacity of the pumping station is given as 300 m^3/s. The main canal is designed for 226,800 hectares [NWRP, 2005].

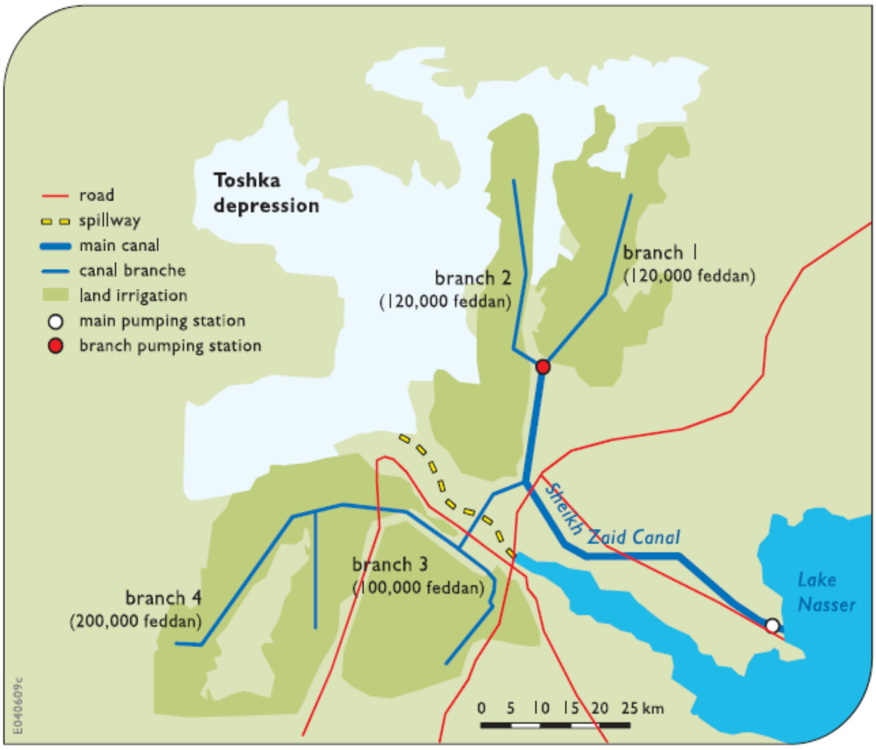


Figure 4.19. Overview of Toshka project [Source: NWRP, 2005].

Information about the main project elements as provided by the Egyptian Ministry of Water Resources and Irrigation (MWRI) are illustrated as follows:

4.7.1 Major Pumping Station (Mubarak Pumping Station)

- The project starts with a giant major pumping station to be setup on the left bank of the AHDR 8 km north of Toshka depression (figure 4.20).
- The station consists of 21 pumps, three of them are standby units.
- The station has been designed to have a maximum static lifting of about 52.50 meters to guarantee its operation when the water level in the AHDR reaches its lowest level of storage, namely 147.5 meters.
- The designed discharge of the pumping station was estimated to be about 300 m^3/s (25 million m^3/day) subject to rise to 428 m^3/s (37 million m^3/day) if necessary.
- Since coming into service, the station has pumped over 162 m^3/sec (14 million m^3/day) of water out of the AHDR.
- The station would be fed with electricity through a transmission station, linked to the electric power line from Aswan.

Figure 4.20. Mubarak Pumping Station [Source: Google, 2009].

4.7.2 The Main Canal (El Sheikh Zayed Canal)

The main canal has the following criteria:

- Canal Length 50 Kilometers.
- Bed Width 30 meters.
- Water Depth 6 meters.
- Side Slope 2 : 1.
- Free Board 1 meter.
- Berm Width 8 meters.
- Bank Width 20 meters.

4.8 THE LOSSES

4.8.1 Evaporation Losses

The Ministry of Water Resources and Irrigation in Egypt "MWRI" for many years adopted the figure of 7.5 mm/d as the annual mean evaporation which correspond to an evaporation rate of 2.70 m/yr. [Whittington & Guariso, 1983].
A later review of previous literature data established a large range for evaporation from AHDR between 1.7 m/yr and 2.9 m/yr [Sadek et al., 1997]. Based on water balance, energy budget and modeling techniques, narrower range of 2.1 m/yr to 2.6 m/yr, with an average of 2.35 m/yr, was calculated by Sadek et al., (1997). In a 2002 technical report, based on the available data at the Nile Forecasting Center in Cairo, it was estimated that the annual evaporation from the AHD Reservoir varied between 12 and 12.6 BCM/yr which correspond to an evaporation rate of 2.0 to 2.1 m/yr [Theodora et al., 2006]. Hassan et al., (2007) computed the evaporation losses from the reservoir and found that the yearly average of the daily evaporation rate is 6.33 mm/day and the average volume of the annual water lost by evaporation is about 12.5 BCM.
The annual evaporation losses from the reservoir are divided between the months, the highest evaporation rates from the reservoir occur in May-October, while the lowest values occur in the period December-February as shown in figure 4.21.
In this simulation analysis, evaporation is calculated monthly as a function of the surface area of the AHDR and fixed monthly coefficients.

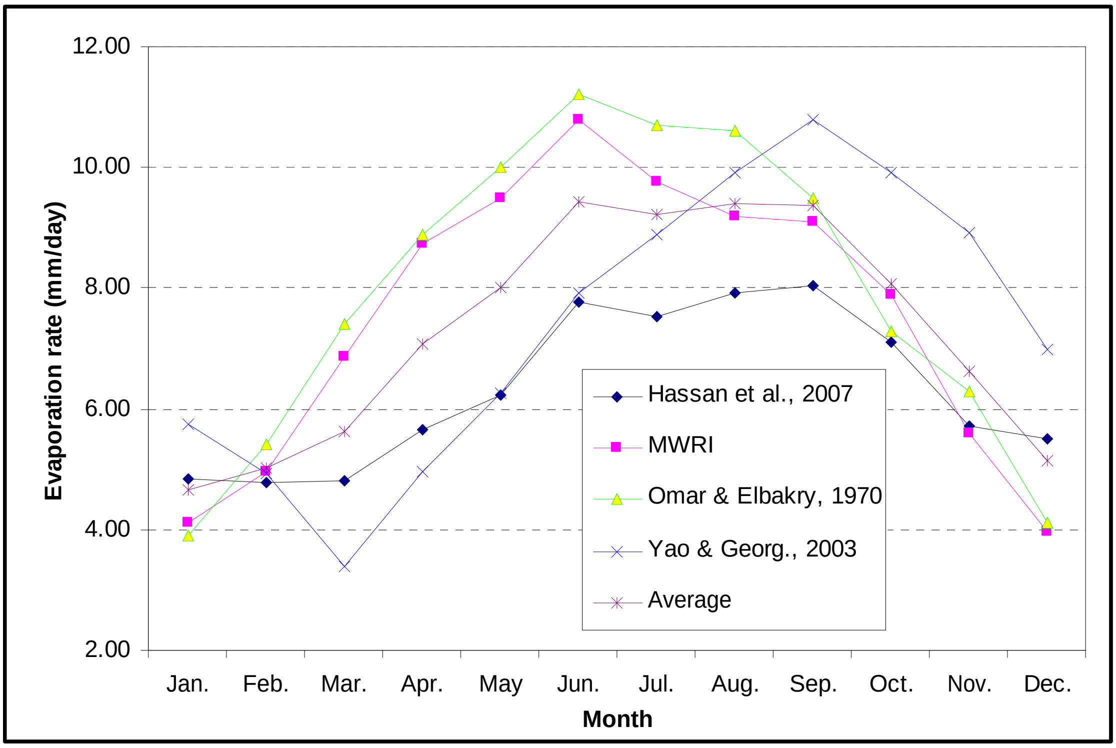


Figure 4.21. Estimates of monthly evaporation rates from the AHDR [Source: Whittington & Guariso, 1983; Yao & Georgakakos, 2003; Hassan et al., 2007].

The computed monthly evaporation coefficients by Hassan et al., (2007) were used in the simulation, because this study showed general agreement with the previous studies for evaporation estimating. Hassan et al., (2007) based on several sets of field data representing the reservoir:

- The Egyptian Meteorological Authority (EMA) operates a network of meteorological stations comprising about 100 stations covering the entire area of Egypt. Of these stations, six are distributed on the surface area of the reservoir upstream the AHD. These stations are operated either by the weather authority or the Aswan High Dam Authority, the locations of these stations with respect to the reservoir boundaries are illustrated in figure 4.22. The actual data of these stations were used in the research to compute the amount of water lost by evaporation from the reservoir.
- Topographical maps for the reservoir representing 20 m interval contour were used to extract the reservoir surface area at different levels. These maps were produced by the Egyptian Survey Authority (ESA) in 1991. They were compiled from aerial photography of 1988. Field surveys were conducted for verification and compilation in 1990.

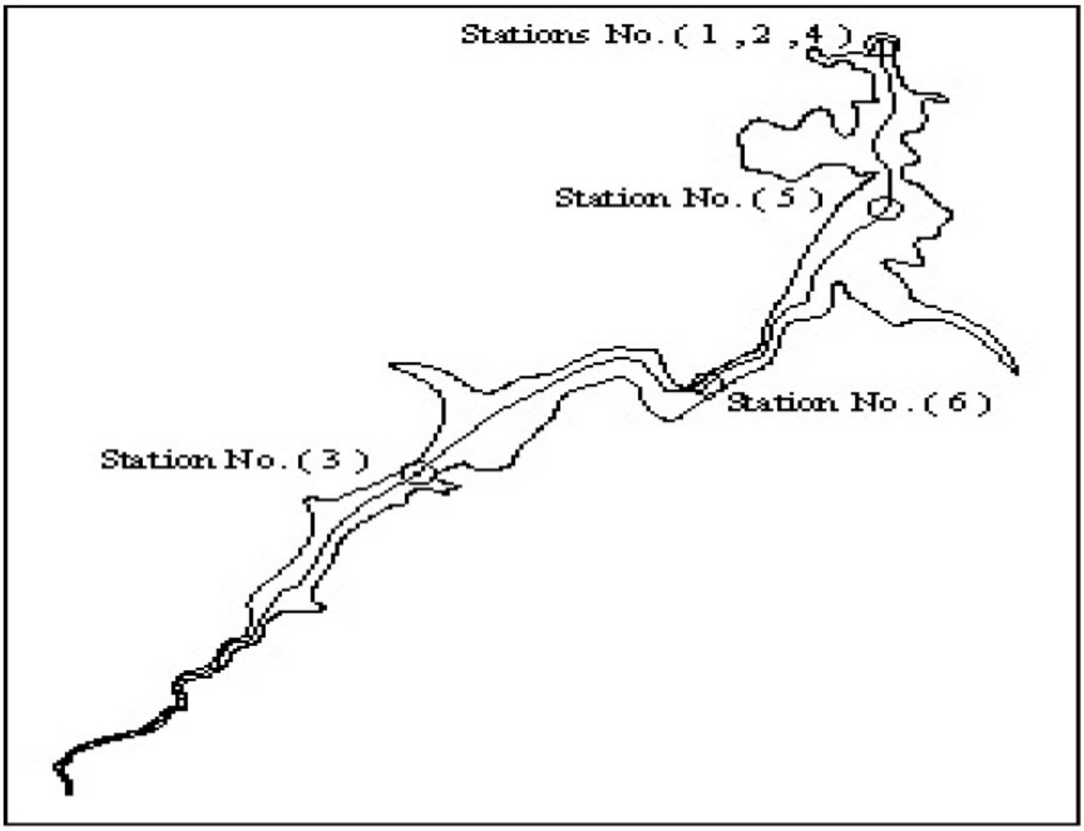


Figure 4.22. Locations of metrological stations over the AHDR [Source: Hassan et al., 2007].

4.8.2 Seepage

The seepage losses were always neglected in the hydrological investigations of the Nile and only the evaporation losses were considered. It should be recognized that the evaporation losses are indeed real losses whereas seepage losses are actually gains to the groundwater reservoirs that can be recovered to supplement surface water if desired. Moreover, water stored from seepage around the AHDR may be very beneficial in the future to improve the complementary between agricultural and power generation. The seepage losses from the AHDR should be considered as a gain of major importance and benefits.

The regional throughflow (seepage) to the groundwater system is relatively small, it amounts to less than 10 percent of the average annual losses [Fahmy, 2001]. At the early stages of planning the AHD, the seepage losses were estimated between 1.0 to 2.0 km^3/yr. at a reservoir level of 180 m [Kashef, 1981]. As illustrated in figure 4.23, the seepage losses are estimated to decrease, as a result of sedimentation within the reservoir basin, from 1.6 BCM/yr in 1975 to approximately 0.7 BCM/yr by the year 2065 [Shenouda et al., 1984]. In the simulation analysis, it was simply assumed that net seepage losses would be 7 percent of the average annual losses.

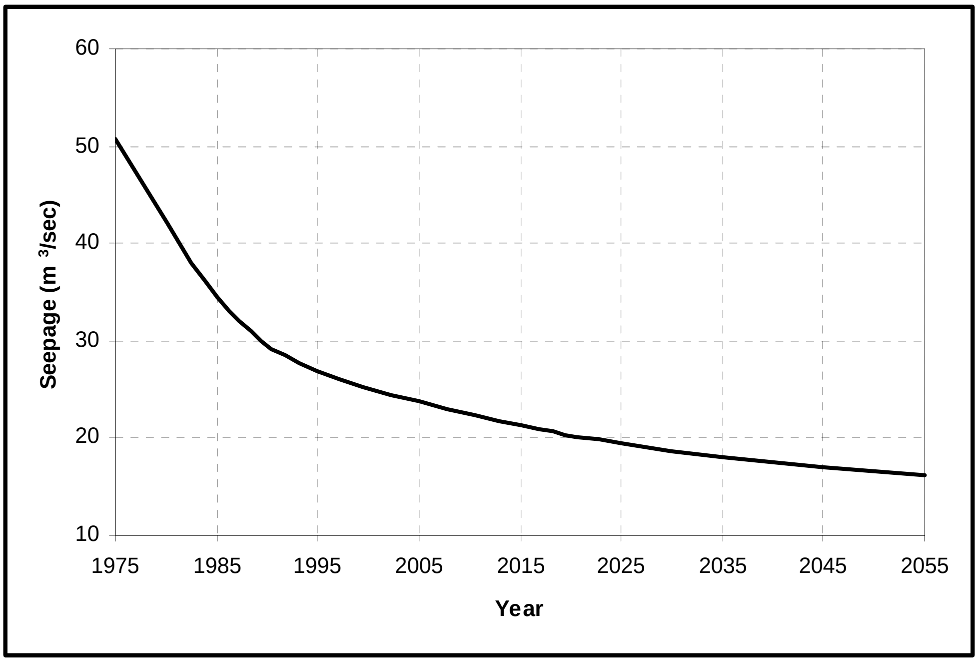


Figure 4.23. Seepage losses [Source: Shenouda et al., 1984].

5 FUTURE SCENARIOS

5.1WHY SCENARIOS – DEFINITIONS

Scenarios represent visions of what may happen in the future, particularly with respect to those factors that are likely to be important in determining the demand-supply conditions. Egypt has to support the socio-economic development objectives, e.g. provide its inhabitants with access to sufficient drinking water of good quality, provide water to farmers to irrigate their lands and provide water to industry. But will the Nile (source of Egypt's water) provide in future more or less water to be distributed among the riparian countries? Can water conservation projects upstream of the AHDR have important impacts on the supply of water for Egypt? And what about climate change? All these uncertainties that are beyond the control of the water resources planners are captured in scenarios. By developing alternative scenarios different possible futures can be analysed, trying to find the best strategy to deal with that future and the uncertainties involved.

5.2 NILE BASIN DEVELOPMENT SCENARIOS

How much water could be made available in addition to the existing amount in the Nile Basin by supply-side projects? Various projects have been discussed over the last hundred years, either concerning where the water is stored to minimize evaporation from reservoirs, or related to building canals through wetlands to reduce evaporation from them. In this study, the scenarios cover a wide range of options from minimal basin development and no cooperative water management to high development and basin-wide cooperation. The following discussion includes some details about these scenarios.

5.2.1 Scenario I "Current Basin Development"

Scenario I represents the existing state of basin development. The lower equatorial lakes are unregulated and there are no sizeable reservoirs along the Upper Blue Nile in Ethiopia. Existing reservoirs include the Owen Falls Dam in Uganda; the Gebel el Aulia, Sennar, Roseires, and Khasm el Girba in Sudan; and the Old Aswan and Aswan High Dams in Egypt (figure 5.1). According to this scenario, Egypt's share of the water available from the Nile is 55.5 BCM/year.

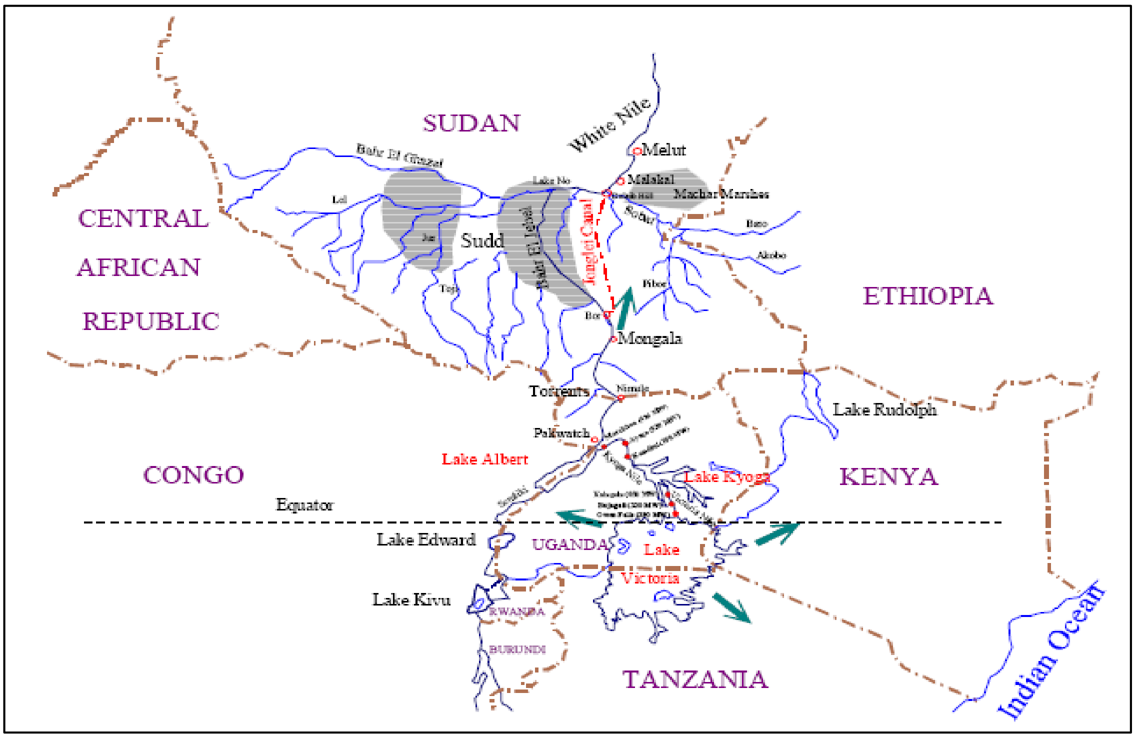


Figure 5.1(a). Southern Nile system with existing and planned development [Source: Tidwell, 2006].

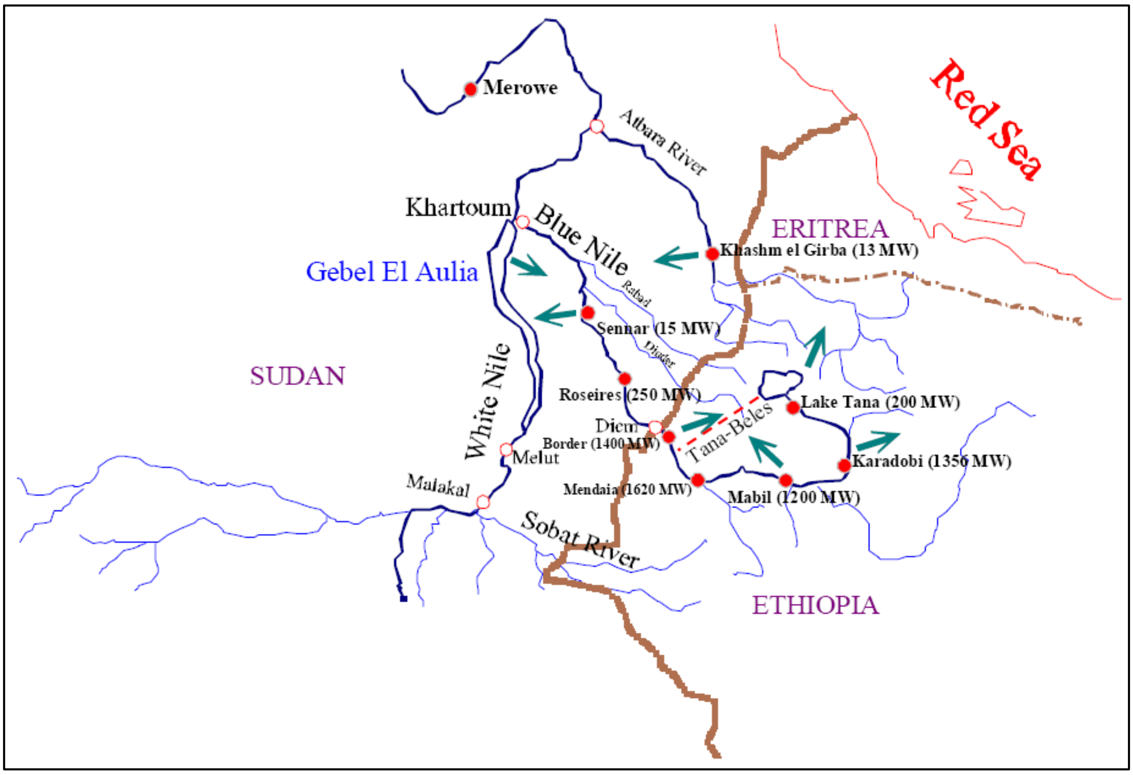


Figure 5.1(b). Eastern Nile system with existing and planned development [Source: Tidwell, 2006].

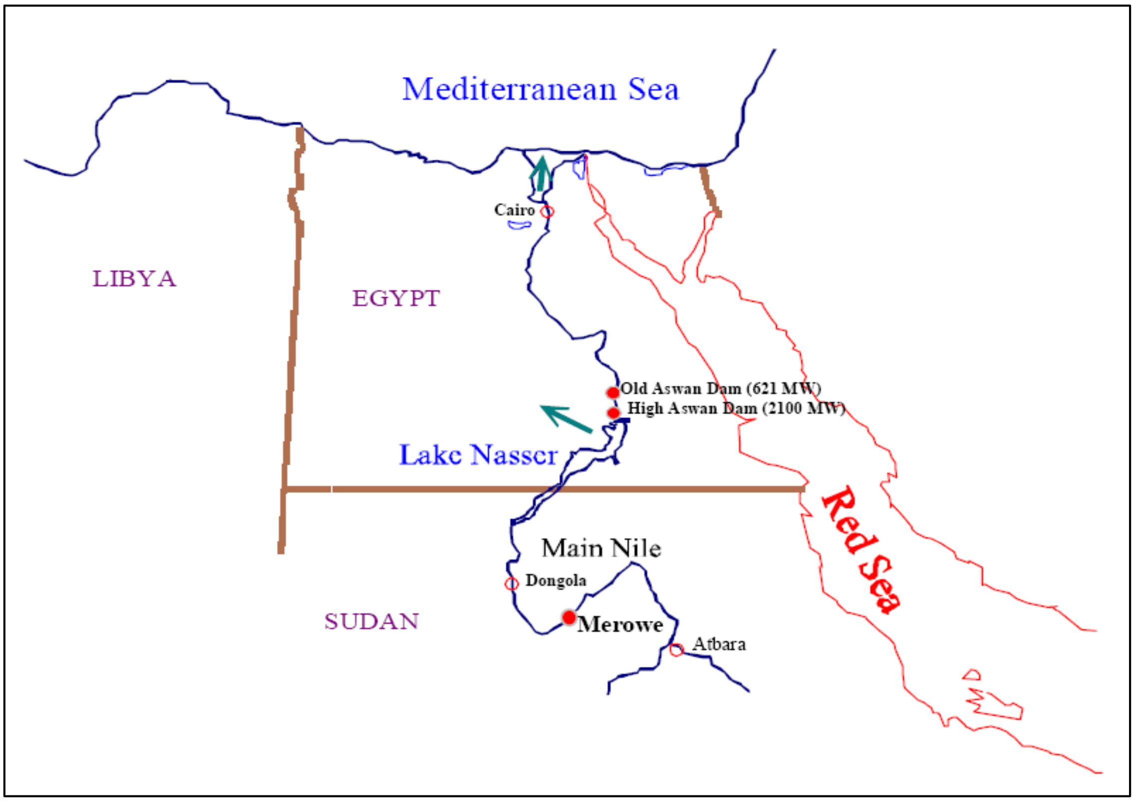


Figure 5.1(c). Main Nile system with existing and planned development [Source: Tidwell, 2006].

5.2.2 Scenario II, III "Jonglei Canal"

In addition to the projects in scenario I, Scenario II and III assume full implementation of Phase I and Phase II of Jonglei Canal project respectively. The Jonglei canal project, considered one of the most important integration projects between Egypt and Sudan. In 2008, the Sudanese and Egyptian governments decided to resume work on the Jonglei Canal project, which had been abandoned for 24 years as a result of the Sudanese civil war. This project in southern Sudan plans to by-pass, and thus drain, part of the wetlands of the Bahr al-Jabal and Bahr az-Zaraf rivers into the White Nile (figure 5.2). The central objective was to increase the Nile revenue by 4.7 BCM annually, measured at Malakal (equivalent to 3.8 BCM measured at Aswan), to be shared equally by the Sudan and Egypt. This would be raised to 7 BCM if Phase II of the project was executed [Ahmed, 2008].
The project involves a canal 280 kilometers long, four meters deep, 52 meters wide, with a 7–9 centimeters/kilometer slope and calculated to deliver an average of 20 million cubic meters of water per day. It has been estimated that this would shrink the permanent marshes by 34–43 per cent. In the second phase of the project, the discharge would be raised to 43 million cubic meters per day, either through an increase in the canal’s cross section or through digging an additional canal [Ahmed, 2008].

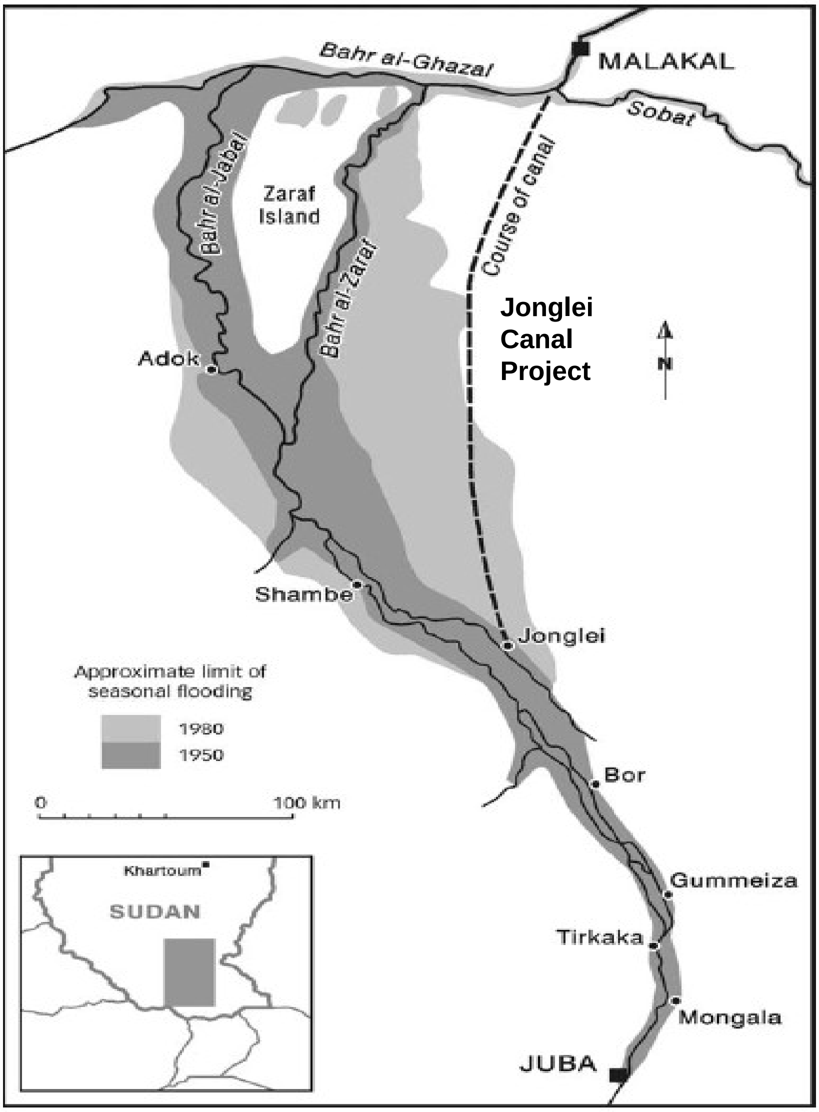


Figure 5.2. The Sudd and the Jonglei Canal [Source: Ahmed, 2008].

5.2.3 Scenario IV "Baro-Akobo Multi- Purpose Water Resources Sub-Project"

In addition to the projects in scenario I, II and III, Scenario IV assumes full implementation of "Baro-Akobo Multi-Purpose Water Resources Sub-Project" in Gambela, South-eastern Ethiopia on the Ethiopian-Sudanese border (figure 5.3). Estimated water savings are about 4 BCM/year. The three countries, Egypt, Sudan and Ethiopia seem to be agreeing on building a canal through the swamps, allocating the additional water to Egypt and Sudan and allocating an equal amount from the Blue Nile river to Ethiopia [Mason, 2003].

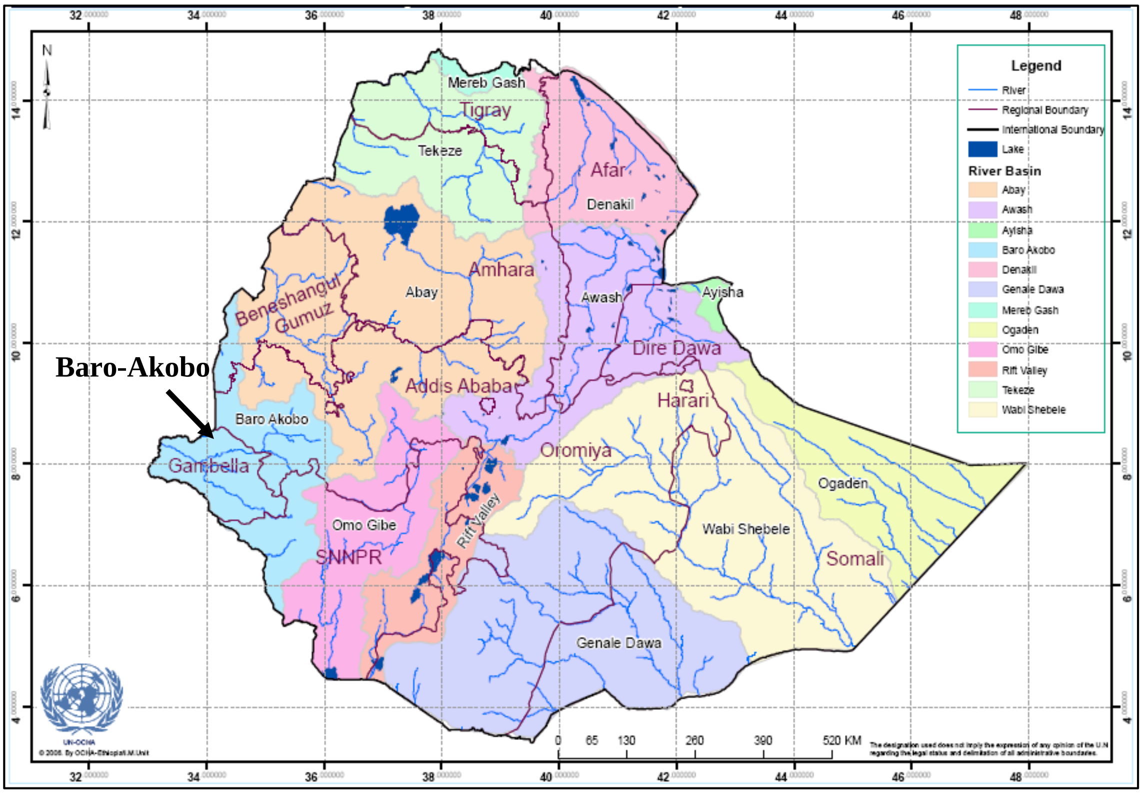


Figure 5.3. Ethiopia River Basins and Baro-Akobo Basin [Source: UN-OCHA-Ethiopia, 2006].

The Baro-Akobo Basin provides a potential opportunity to develop and manage a multi-purpose water resources project which may provide win-win benefits to the Eastern Nile countries. The area, though currently poor, relatively undeveloped, and subject to erosion and land degradation, has plentiful land and water resources. There is a substantial untapped potential for hydropower development, and opportunities for developing irrigation as well as improving rainfed agriculture. Portions of the basin which are subject to extensive flooding and high evaporation and seepage rates could potentially yield important conservation gains. The area contains natural assets, such as wetland and wildlife areas. Single purpose projects to address any of these opportunities, however, would have limited benefits. Development of multi-purpose water resources and associated rural development investments, however, could optimize gains and provide transboundary benefits. Water resources infrastructure which provides storage and river regulation, particularly if coupled with non-structural measures and socio-economic development activities, could provide opportunities for agricultural production, water conservation, navigation, fisheries, environmental management, flood and drought mitigation, and hydropower, providing the economic growth for substantial improvement of livelihoods for the local population as well as broader socioeconomic benefits for the region [ENSAP, 2007].

5.3 CLIMATE CHANGE SCENARIOS

Beyene et al. (2009) assessed the potential impacts of climate change on the hydrology and water resources of the Nile River basin using a macroscale hydrologic model driven by 21st century simulations of temperature and precipitation downscaled from runs of 11 General Circulation Models (GCMs) and two global emissions scenarios (A2, corresponding roughly to unconstrained growth in emissions, and B1, corresponding to elimination of global emissions increases by 2100) archived for the 2007 IPCC report (figure 5.4).

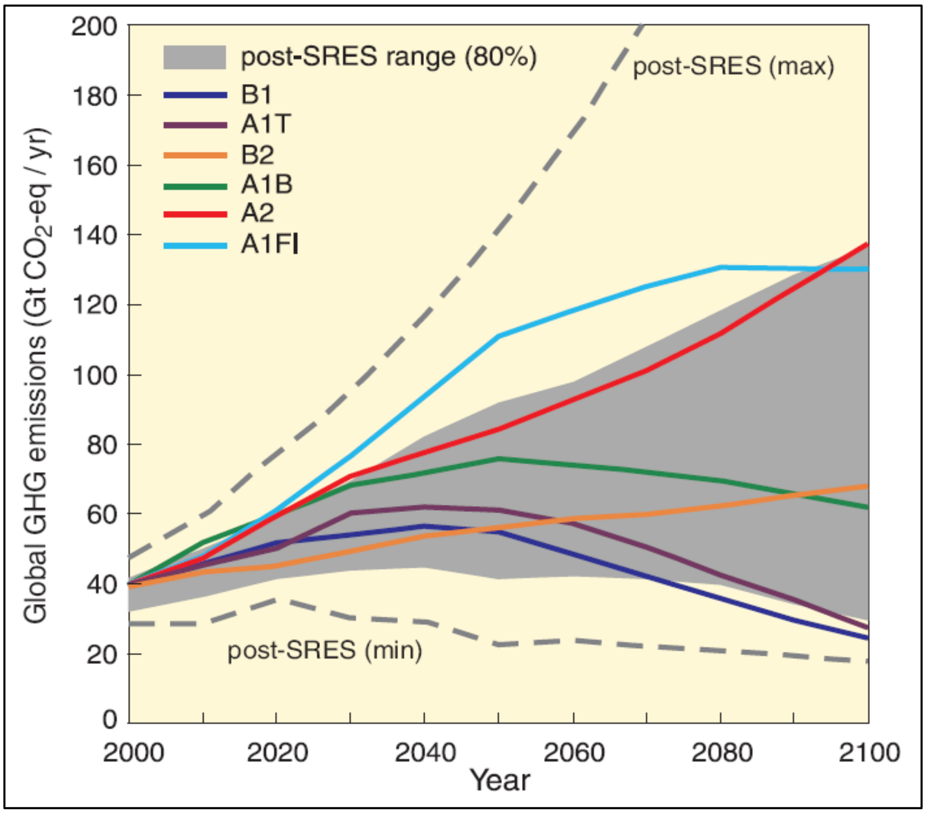


Figure 5.4. Scenarios for global GHG emissions from 2000 to 2100 in the absence of additional climate policies [Source: IPCC, 2007].

The results show that, averaged across the multimodel ensembles, the entire Nile basin is expected to increases in precipitation early in the century (period I, 2010-2039), followed by decreases later in the century (period II, 2040-2069 and period III, 2070-2099) with the exception of the eastern-most Ethiopian highlands which might experience increases in summer precipitation by 2080-2100.

Averaged over all models and ensembles, annual streamflow at the AHD for scenario A2 was predicted to increase to 111% of the 1950-99 mean during 2010-2039, but then to decrease to 92% and 84% of the 1950-99 mean during 2040-2069 and 2070-2099, respectively. For scenario B1, the corresponding numbers were 114% (increase) during 2010-2039, and decreases to 93% and 87% of the 1950-99 mean from 2040-2069 and 2070-2099, respectively.

The climate change scenarios generated by Beyene's study will be used as a multiplier to the historical natural series (1965-1994) to the model for simulation future inflows to the reservoir.

5.4WATER DEVELOPMENT AND MANAGEMENT SCENARIOS

The water development scenarios and the related water demand targets are illustrated and presented in the tables 5.1 and 5.2 respectively. Figure 5.5 shows the summary of the average annual flow scenarios at Dongola (the AHDR entrance) for the basin development and climate change scenarios (baseline (1965-1994), period I (2010-2039), period II (2040-2069), and period III (2070-2099)).

	Current Condition	Jonglei Canal		Baro-Akobo Project
		Phase I	Phase II	
Scenario I	Yes			
Scenario II	Yes	Yes		
Scenario III	Yes	Yes	Yes	
Scenario IV	Yes	Yes	Yes	Yes

Table 5.1. Nile basin development scenarios.

Scenario I	Scenario II	Scenario III	Scenario IV
55.50	57.40	59	63

Table 5.2. Egypt's water withdrawal Targets (BCM/year).

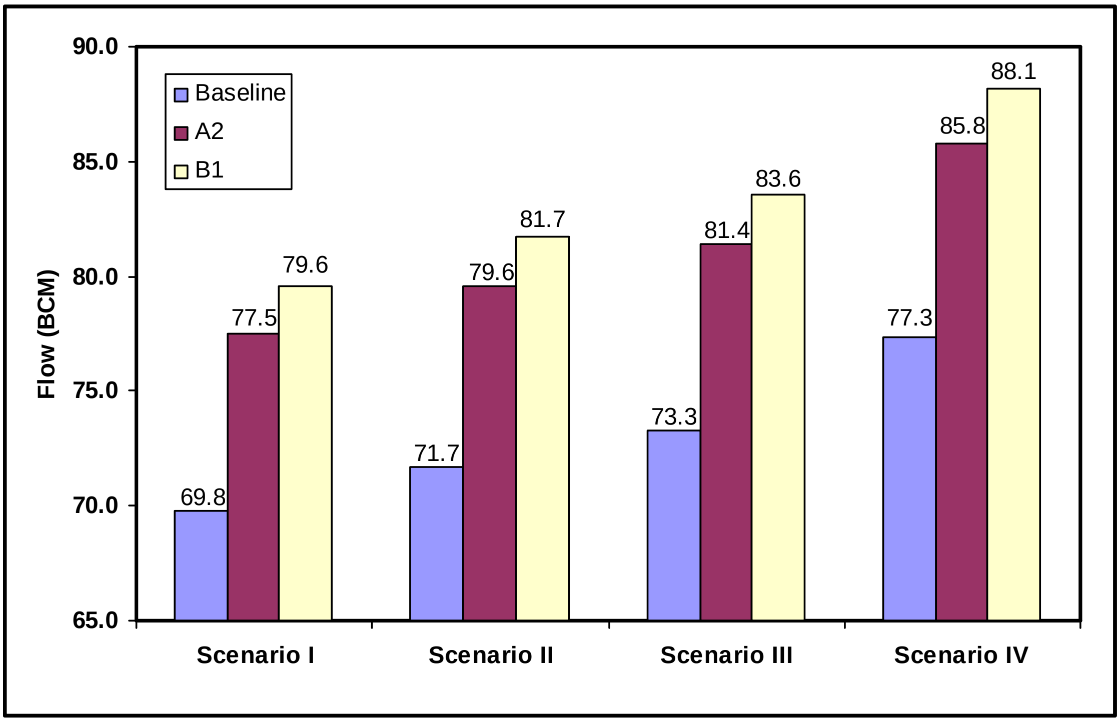


Figure 5.5(a). Average annual flow scenarios at Dongola during period I (2010-2039).

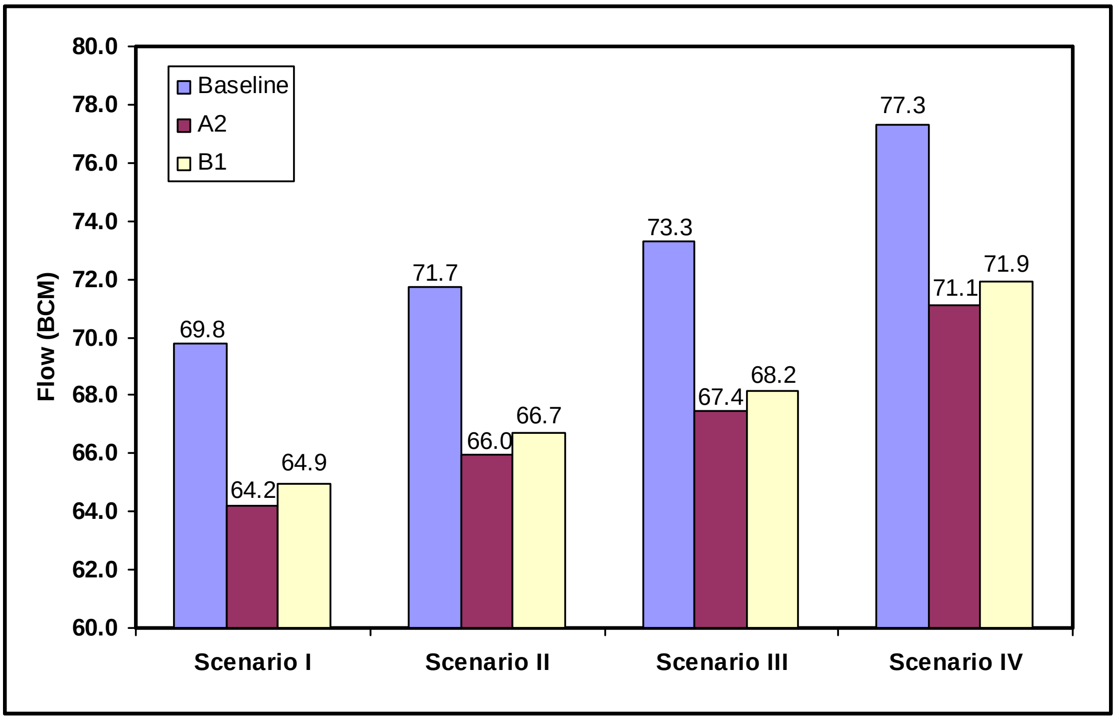


Figure 5.5(b). Average annual flow scenarios at Dongola during period II (2040-20639).

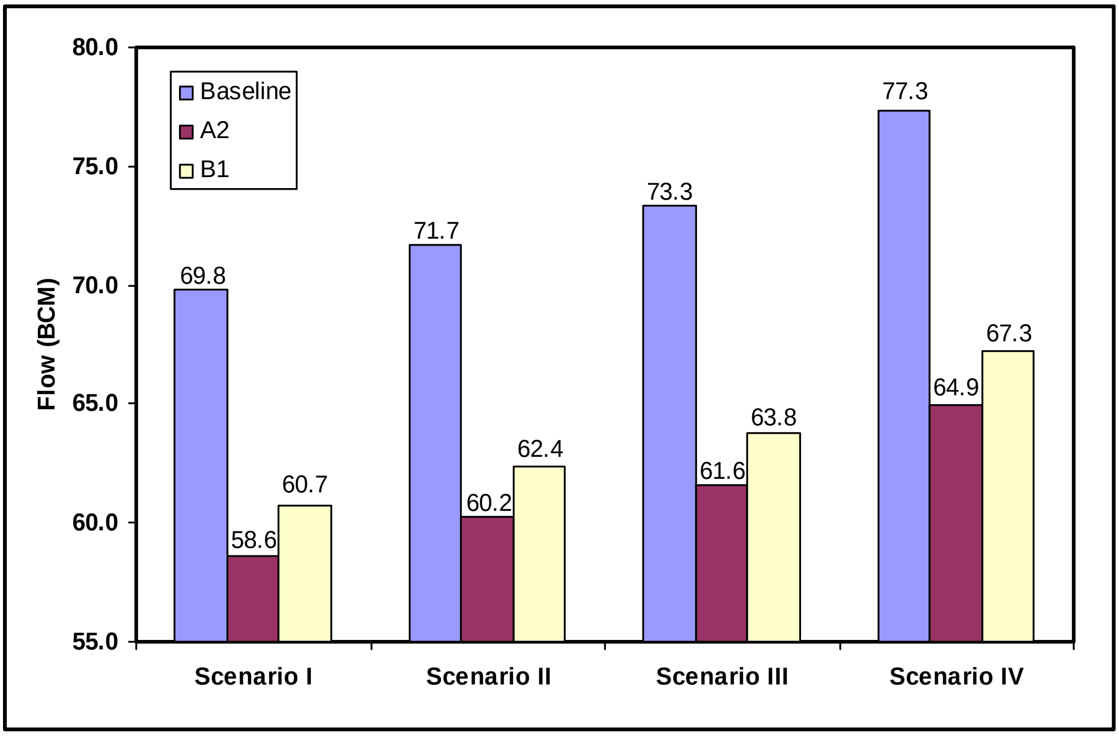


Figure 5.5(c). Average annual flow scenarios at Dongola during period III (2070-2099).

6 SCENARIO ASSESSMENTS

6.1 PROCEDURE OF THE SCENARIOS ANALYSIS

The future hydrologic scenarios developed have been used to assess the expected impacts to potential climate change and basin development scenarios. The operation polices which were determined by the Ministry of Water Resources and irrigation (MWRI) were used for the scenarios analysis in the simulation model of the AHDR.
The current operation polices aim to lower the level of the reservoir to at least 175 meters by approximately July 31 each year (i.e., before the arrival of the flood), in order for the reservoir to have the capacity to store the peak of a high flood. If the level of the reservoir is at 175 m on July 31 (it could, of course, be lower after a series of low and average years), the entire incoming flood and all the subsequent inflows of the water year minus the evaporation and seepage losses must be released over a twelve month period in order to bring the water level back down to 175 m by the following July 31.
The monthly discharges from the AHD follow a fixed pattern of releases (figure 6.1, approximately the 1982 discharge program of Aswan High Dam Authority) unless more water must be released over the water year (August 1-July 31) in order for the reservoir level to be down to 175 m by July 31. This discharge program is assumed to satisfy all water supply objectives. Any water in excess of this discharge program is evenly distributed in the months with the lower water requirements in such a way that the peak monthly discharges are reduced as much as possible.
For example, consider two cases illustrated in figure 6.1. If the operating rule required the releases of, for example, 90 BCM during a high flood. The assumed operation policy would discharge 7.50 BCM each month throughout the year. If a discharge of 70 BCM was required, the excess of 14.50 would be allocated among the months with low water requirements. The monthly discharges during the peak summer months remain unchanged. The months with low water requirements receive additional releases, but each such month does not receive the same amount. The months with the lowest requirements receive a greater volume of the excess water in order that all months in which any additional water is spilled have the same total discharge.

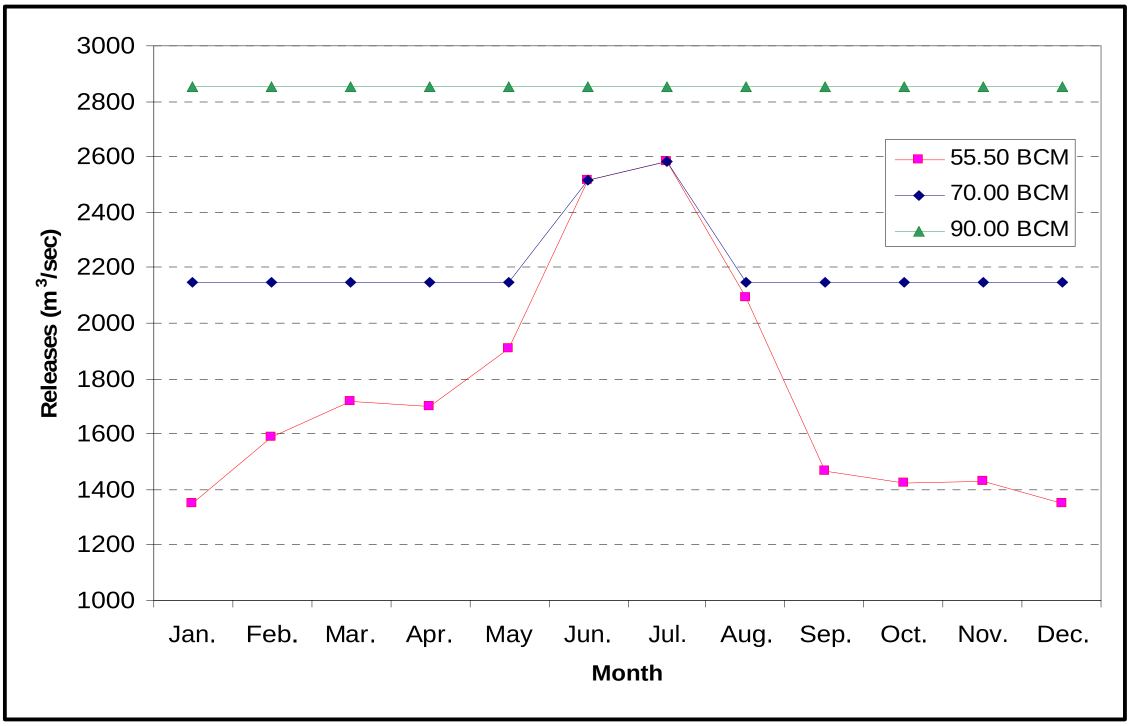


Figure 6.1. Fixed releases program and two example of assumed operating policy during years of high flood and high reservoir levels.

The application of this operating rule is only possible when the total inflows over the water year plus the possible variation of the storage exceed the sum of the discharges. If this condition does not hold, the simulation model uses the sliding scale for reduction which the Ministry of Water Resources and Irrigation (MWRI) suggested for coping with a series of low floods. The reductions in withdrawals should being if the reservoir contents in the live storage zone were less than 60 BCM on July 31, the details of the sliding scale for reductions are presented in table 6.1.

Live Storage Contents (S) on July 31	**Reduction percentage**
S≥ 60	full share
55 ≤ S < 60	5 %
50 ≤ S < 55	10 %
S < 50	15 %

Table 6.1. Sliding scale for reduction [Source: MWRI, 2004].

6.2 SCENARIO ASSESSMENTS

In the following discussion, the assessment results are summarized relative to the following criteria:

- Water supply releases.
- Reservoir level variations.
- Hydropower production.
- Evaporation losses.
- Discharges to Toshka spillway.

The following sections are an executive summary of the assessment results. The presentation focuses on monthly and annual average quantities and includes four sections associated with the four previously defined development and management scenarios. Each section evaluates the impacts of climate and demand change on the various water uses. More detailed information on the entire cumulative frequency of the results is provided and discussed in Appendix A.
For purposes of comparison, each chart presented in this assessment features a baseline. The baseline series represent simulated basin response under the baseline climate but for a given development scenario. This allows direct comparison of the system sensitivity to both climate change and basin development. Graphical results are presented by (1) development scenario, (2) climate scenario (baseline and three periods with two global emission scenarios A2 (B1)).

6.2.1 Development Scenario I

This scenario represents the current basin condition with no further development, Egypt is entitled to withdraw 55.5 BCM/year.

6.2.1.1 Sensitivity of water supply releases to climate change

In general the mean annual withdrawal from the AHDR for the three periods (period I (2010-2039), period II (2040-2069), and period III (2070-2099)) and two global emission scenarios A2 (B1) are 61.56 (63.33), 52.70 (53.2) and 50.42 (51.32) BCM, respectively, compared to the baseline release of 56 BCM (figure 6.2(a)).
Under baseline climate scenario, Egypt falls short of its target demand in approximately 31 % of years. This percentage decreases to 10 (7) % of years for the period I A2 (B1) emissions scenario, and increases to 45 (41) % of years for the period II A2 (B1). In contrast, during period III this percentage grows significantly to reaches to 62 (55) % of years for two global emission scenarios A2 (B1). (figure A.1(a)).
In this scenario there is a significant probability, the withdrawal from the AHDR stops during period III A2 for two months due to the reservoir levels decreasing as a result of the serious reduction in the amount of water entering the reservoir
Maximum spills over and above the fixed discharge program occur in period I (B1) in approximately 52 % of years, in 69 % of this cases the spills are greater than 10 BCM. Also in this period the maximum release downstream the AHD exceeds the allowable maximum releases (2890 m^3/s (250 MCM/day)) to reaches to 3698 m^3/s (319.5 MCM/day) (figure A.1(b,c)).

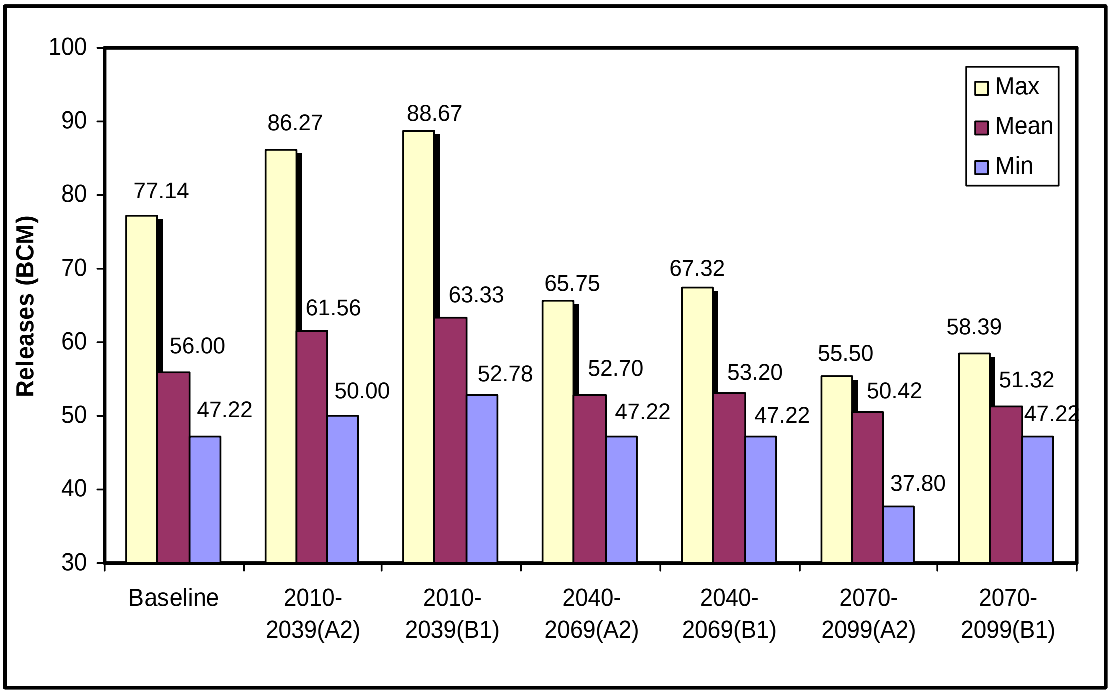


Figure 6.2(a). Annual withdrawal from the AHDR for scenario I.

6.2.1.2 Sensitivity of reservoir level variations to climate change

Figures A.1(d) and A.1(e) present the water level variation projected in the AHDR for baseline and three periods with two global emission scenarios A2 (B1). From these figures, it can be concluded that the water levels upstream the dam are affected by the changes in the inflows. The maximum water levels upstream the AHD of all climate scenarios within the water year did not exceed 182 m, and the minimum water levels are above the minimum allowable limits (147 m). Table 6.2 is a summary characterization of variation in the reservoir level, this analysis represents in the water levels limits and corresponding percentage of occurrence for all climate scenarios.

Level (m)	Baseline	Period I		Period II		Period III	
		A2	B1	A2	B1	A2	B1
	%	%	%	%	%	%	%
> 181	0.30	0.60	0.60	-	-	-	-
> 178	19	27	29	10	11	1	3
> 175	51	75	77	39	39	18	28
> 160	100	100	100	97	98	76	88
< 150	-	-	-	-	-	2	-

Table 6.2. Level variations characteristics in the AHDR for scenario I.

6.2.1.3 Sensitivity of hydropower production to climate change

Figure 6.2(b) shows a wide spread in the average annual hydropower production at the AHD for the A2 and B1 global emission scenarios. Under the baseline climate scenario, annual hydropower production at the AHD varies between 5572 - 10641 GWh, with a mean of about 7570 GWh. The annual average power production at the AHD generally follows changes in streamflow, increasing early in the century to 113 (117) percent of baseline production for the period I A2 (B1) emissions scenario, but then decreasing to 91 (92) and 80 (85) percent of the baseline mean for Periods II and III, respectively.

There are a few aspects of the frequency distributions that are also notable, maximum annual hydropower production occurs in period I (B1), and exceed 10000 GWh in approximately 17 % of years. For period III A2, the hydropower production stops for two months due to the reservoir levels falling below the minimum level for the hydropower generation, and the annual hydropower production is less than 8000 GWh in almost all years (figure A.1(f)).

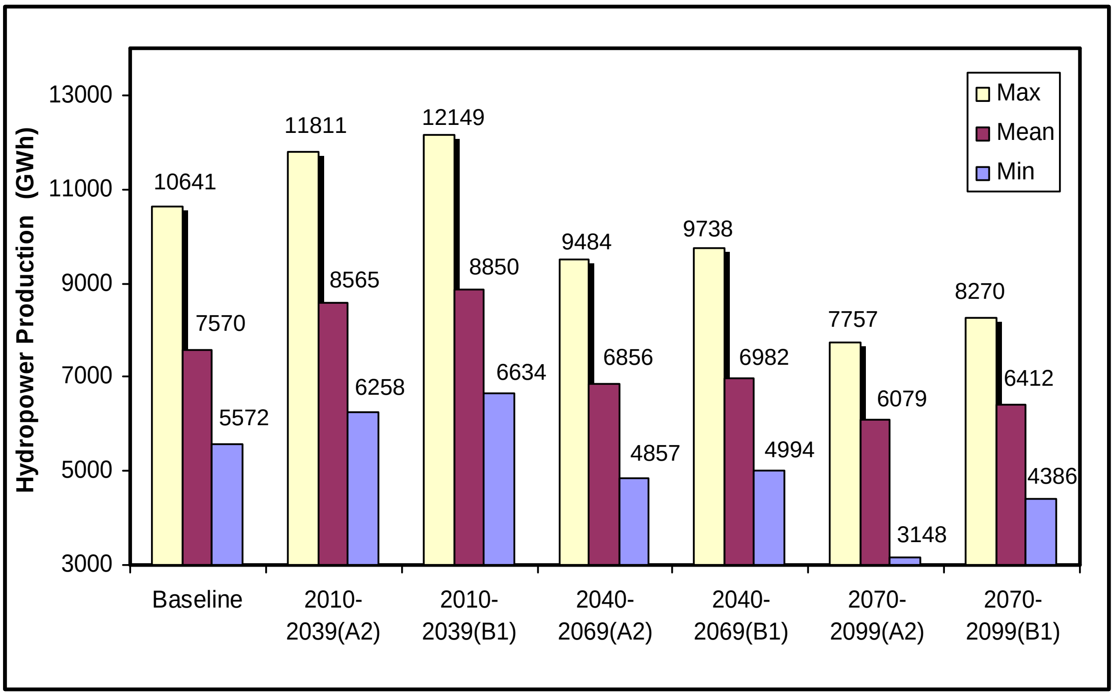


Figure 6.2(b). Annual hydropower production at the AHD for scenario I.

6.2.1.4 Sensitivity of evaporation losses to climate change

According to climate change scenarios, the annual evaporation losses vary between 8.73 – 13.31 BCM, with a mean of about 11.64 BCM for the baseline climate scenario. Due to streamflow increasing early in the century, the annual average evaporation losses increases to 12.38 (12.47) BCM for the period I A2 (B1) emissions scenario, but then decreases to 10.69 (10.88) and 9.21 (9.89) BCM for Periods II and III, respectively (figure 6.2(c)). Figure A.1(g) illustrates the frequency distribution of the annual evaporation losses, from this figure it can be noticed that 55

(59) and 27 (28) percent of years during periods I, II had evaporation losses greater than 12.5 BCB, compared to the baseline percent of 38 %, while in almost all years during period III had evaporation losses less than 12.5 BCB.

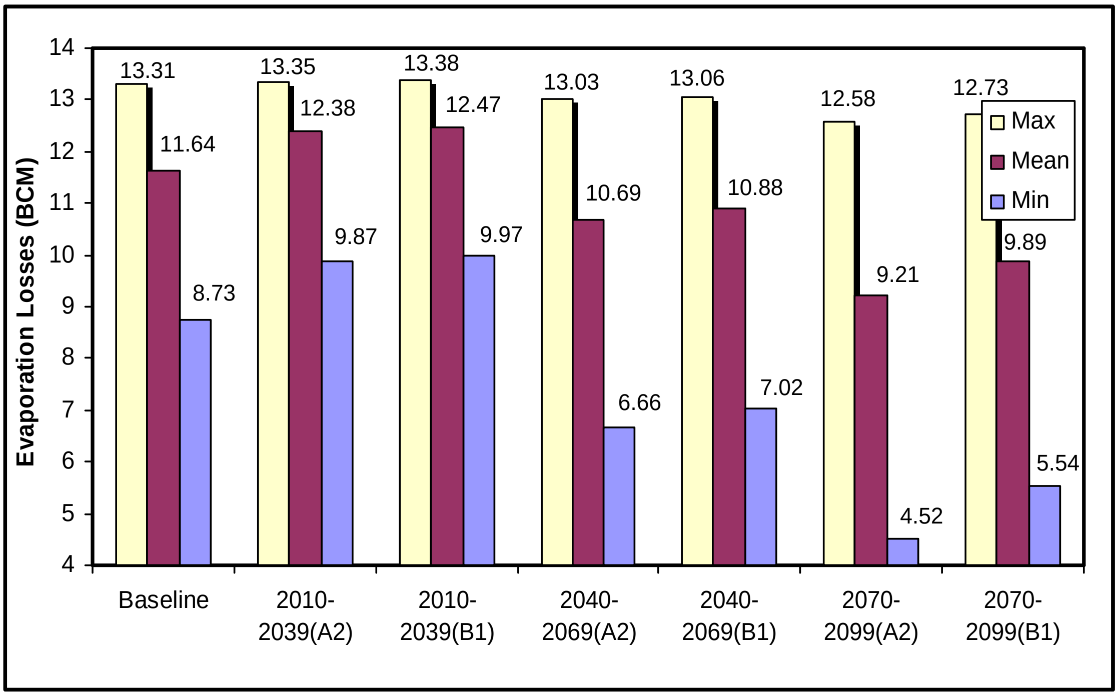


Figure 6.2(c). Annual evaporation losses for scenario I.

6.2.1.5 Sensitivity of Toshka spillway discharges to climate change

Under the baseline climate scenario, annual discharges to Toshka spillway vary between 0.82 – 9.98 BCB with a mean of about 3.28 BCM and occur in approximately 41 % of years (figure 6.2(d)). The outflows discharged to Toshka Spillway are influenced by increasing the annual inflow and raising water level upstream the AHD. For example, discharges to Toshka spillway are negligible in period III due to reduction of the reservoir water levels in this period. For the period II, the range of projected discharges to Toshka spillway is 0.01 (0.17) to 5.71 (6.01) BCM and occur in approximately 38 (38) % of years for A2 (B1) emissions scenario (figure A.1(h)). The majority of scenarios show significant potential increasing in a mount of the released water to Toshka spillway for the period I, the range of projected releases grows to 0.07 (0.12)-12.03 (12.50) BCM and occur in approximately 69 (72) % of years for A2 (B1), respectively.

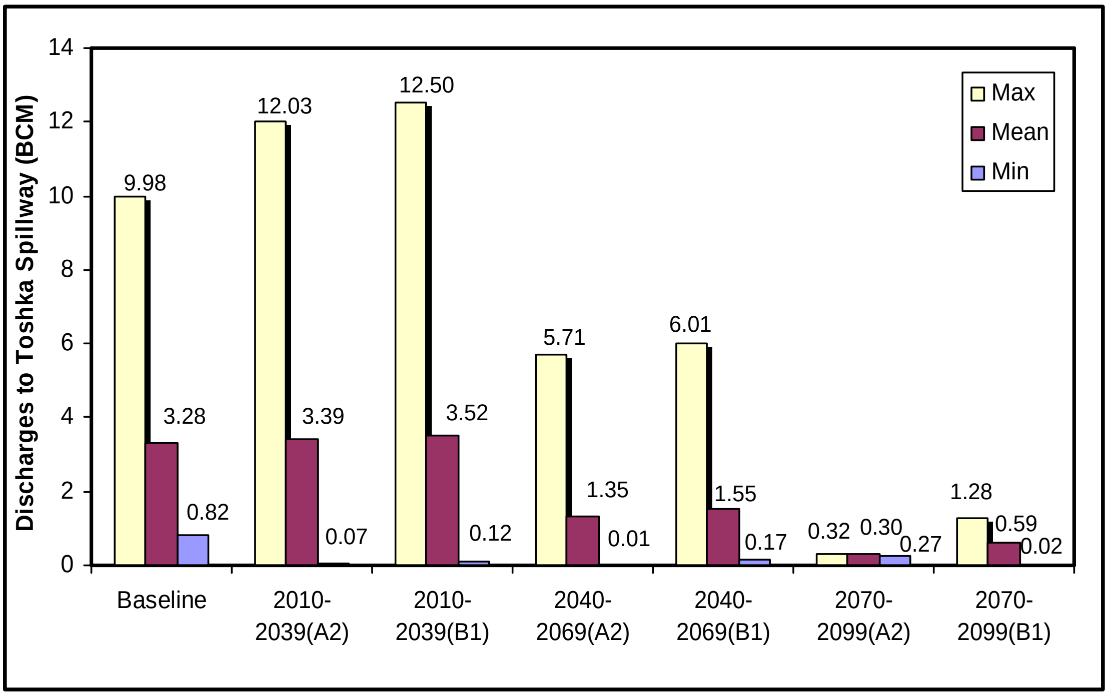


Figure 6.2(d). Annual Discharges to Toshka spillway for scenario I.

6.2.2 Development Scenario II

This scenario includes besides the current basin condition, full implementation of the first stage of Jonglei canal, and Egypt's annual fixed discharges are increased to 57.40 BCM.

6.2.2.1 Sensitivity of water supply releases to climate change

Figure 6.3(a) presents the water releases projected to occur if the first stage of Jonglei canal is implemented. The curve shows improvements in the average annual releases from the AHDR, the mean annual withdrawal for the three periods (period I (2010-2039), period II (2040-2069), and period III (2070-2099)) and two global emission scenarios A2 (B1) are 63.79 (65.47), 54.64 (54.94) and 52.01 (53.13) BCM, respectively, compared to the baseline of 57.91 BCM

There are a few aspects of the frequency distributions (figure A.2(a)) that are also notable, the amount of minimum annual withdrawal from the AHDR increased from 37.8 BCM in scenario I during period III A2 to reaches to 39.10 BCM in scenario II for the same period, and occurs in one year which presents approximately 3.40 % of years.

As a result of the serious reduction in the amount of water entering the reservoir, the withdrawal from the AHDR stops during period III A2 for two months.

In period I (B1), the maximum release downstream the AHD reaches to 4114 m^3/s (355.4 MCM/day) (figure A.2(b,c)). Also in this period, maximum spills over and above the fixed discharge program (57.40 BCM/year) occur in approximately 52 % of years, in 69 % of this cases the spills are greater than 10 BCM.

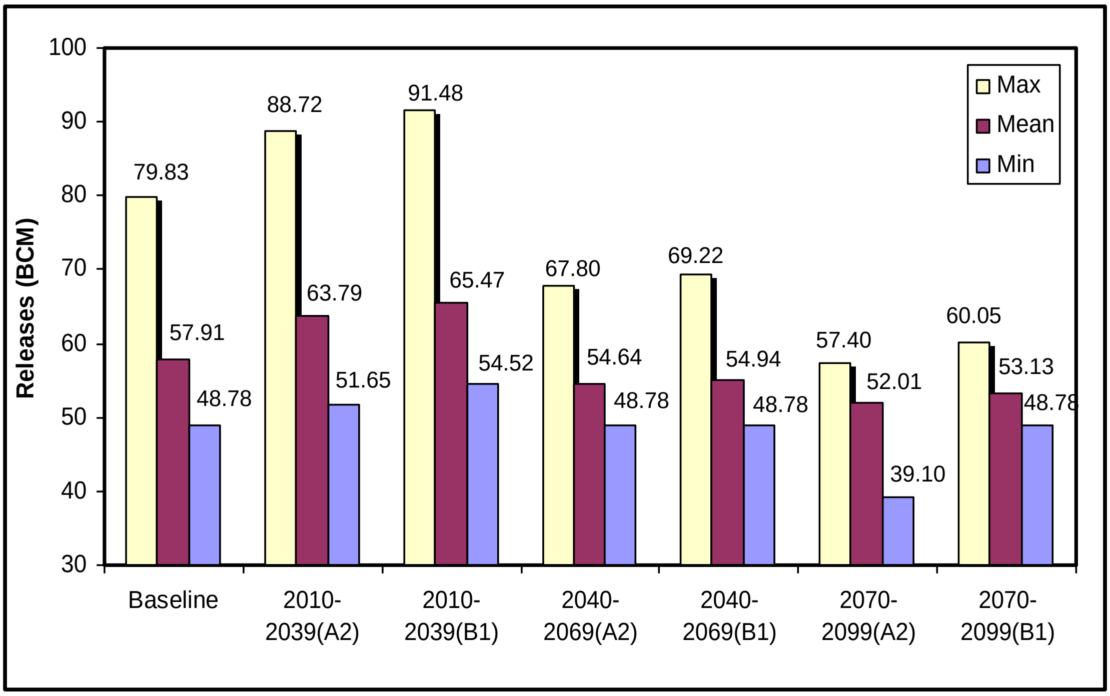


Figure 6.3(a). Annual withdrawal from the AHDR for scenario II.

6.2.2.2 Sensitivity of reservoir level variations to climate change

It can be concluded from figure A.2(d) that the maximum water levels upstream the AHD of all climate scenarios within the water year did not exceed 182 m, and the minimum water levels are above the minimum allowable limits (147 m). Figure A.2(e) illustrates the frequency distribution of the levels in the reservoir, from this figure it can be noticed that the water levels limits and corresponding percentage of occurrence for scenario II are similar to scenario I for baseline and three periods with two global emission scenarios A2 (B1). Therefore, it is expected that the evaporation losses as well as the discharges to Toshka spillway in this scenario are very similar to the values in the previous scenario.

6.2.2.3 Sensitivity of hydropower production to climate change

Under the baseline climate scenario, annual hydropower production at the AHD varies between 5818 - 11055 GWh, with a mean of about 7875 GWh. The annual average power production at the AHD generally follows changes in streamflow, increasing early in the century to 113 (117) percent of baseline production for the period I A2 (B1) emissions scenario, but then decreasing to 91 (92) and 80 (85) percent of the baseline mean for Periods II and III, respectively (figure 6.3(b)).

For period III A2, the hydropower production stops for two months due to the reservoir levels falling below the minimum level for the hydropower generation, and the annual hydropower production is less than 8000 GWh in more than 90 % of years. The maximum annual hydropower production occurs in period I (B1), and exceed 10000 GWh in approximately 24 % of years. (figure A.1(f)).

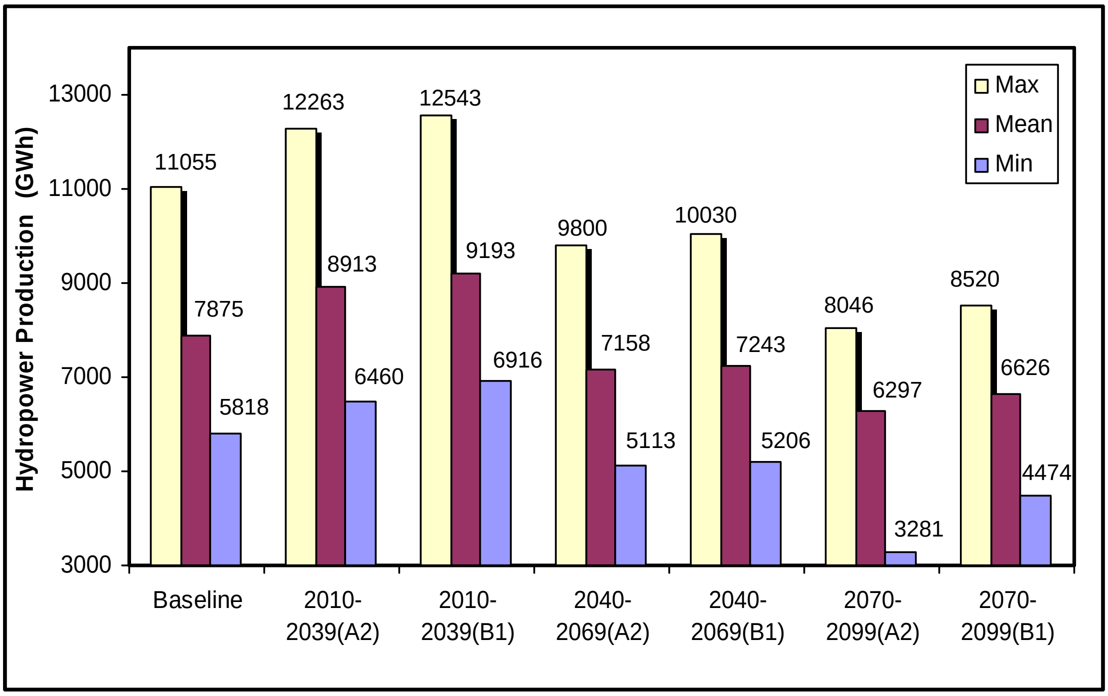


Figure 6.3(b). Annual hydropower production at the AHD for scenario II.

6.2.2.4 Sensitivity of evaporation losses to climate change

Figure 6.3(c) shows the annual evaporation losses for the baseline and three periods with two global emission scenarios. Under the baseline climate scenario, the annual evaporation losses vary between 8.84 – 13.27 BCM, with a mean of about 11.66 BCM. The annual average evaporation losses increases to 12.37 (12.49) BCM for the period I A2 (B1) emissions scenario, but then decreases to 10.75 (10.89) and 9.20 (9.77) BCM for Periods II and III, respectively. It can be noticed from figure A.2(g) that 55 (62) and 24 (31) percent of years during periods I, II had evaporation losses greater than 12.5 BCB, compared to the baseline percent of 38 %, while in almost all years during period III had evaporation losses less than 12.5 BCB.

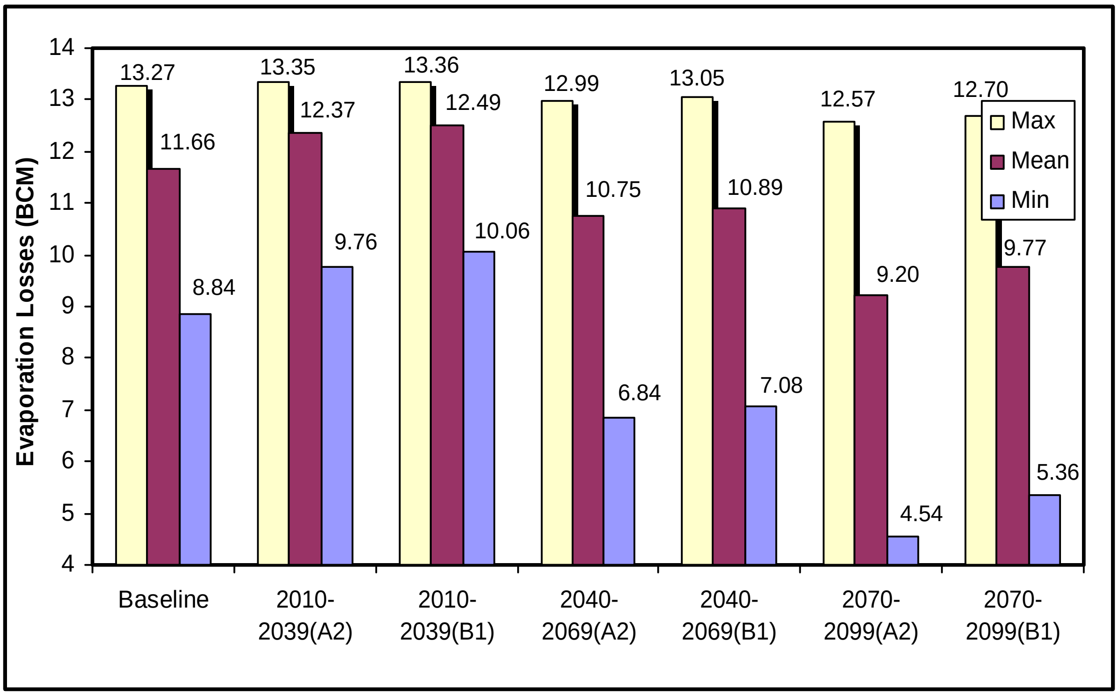


Figure 6.3(c). Annual evaporation losses for scenario II.

6.2.2.5 Sensitivity of Toshka spillway discharges to climate change

Annual discharges to Toshka spillway vary between 0.65 – 9.20 BCB with a mean of about 3.04 BCM and occur in approximately 41 % of years for the baseline climate scenario (figure 6.3(d)). Discharges to Toshka spillway are negligible in period III due to reduction of the reservoir water levels in this period. For the period II, the range of projected discharges to Toshka spillway is 0.06 (0.14) to 5.08 (5.87) BCM and occur in approximately 34 (38) % of years for A2 (B1) emissions scenario (figure A.2(h)). The majority of scenarios show significant potential increasing in a mount of the released water to Toshka spillway for the period I, the range of projected releases grows to 0.06 (0.21)-11.57 (11.96) BCM and occur in approximately 69 (72) % of years for A2 (B1), respectively.

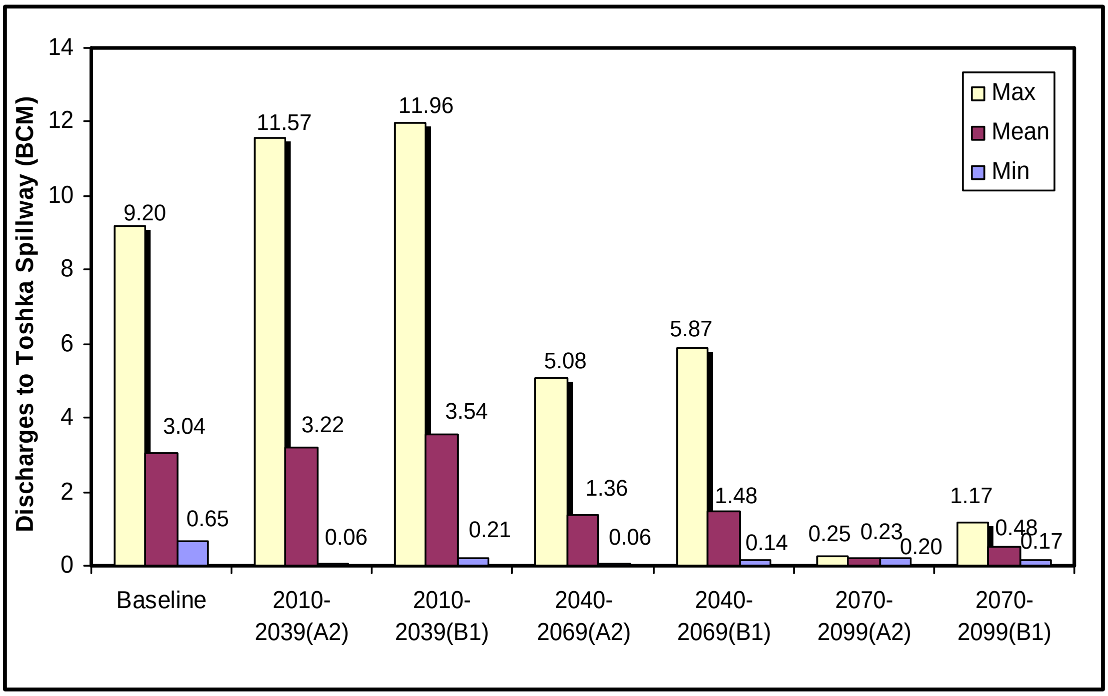


Figure 6.3(d). Annual Discharges to Toshka spillway for scenario II.

6.2.3 Development Scenario III

In this scenario, the second stage of the Jonglei canal is finished and Egypt is entitled to withdraw 59 BCM/year.

6.2.3.1 Sensitivity of water supply releases to climate change

In general the mean annual withdrawal from the AHDR for the three periods (period I (2010-2039), period II (2040-2069), and period III (2070-2099)) and two global emission scenarios A2 (B1) are 65.65 (67.28), 56.11 (56.43) and 53.46 (54.38) BCM, respectively, compared to the baseline release of 59.68 BCM (figure 6.4(a)).
Under baseline climate scenario, Egypt falls short of its target demand (59 BCM/year) in approximately 28 % of years. This percentage decreases to 7 (7) % of years for the period I A2 (B1) emissions scenario and increases to 45 (41) % of years for the period II A2 (B1). In contrast, during period III this percentage grows significantly to reaches to 62 (59) % of years for two global emission scenarios A2 (B1). (figure A.3(a)).
Also, in this scenario there is a significant probability, the withdrawal from the AHDR stops during period III A2 for two months due to the reservoir levels decreasing as a result of the serious reduction in the amount of water entering the reservoir
Maximum spills over and above the fixed discharge program occur in period I (B1) in approximately 55 % of years, in 53 % of this cases the spills are greater than 10 BCM. Also in this period

the maximum release downstream the AHD exceeds the allowable maximum releases (2890 m^3/s (250 MCM/day)) to reaches to 4213 m^3/s (364 MCM/day) (figure A.3(b,c)).

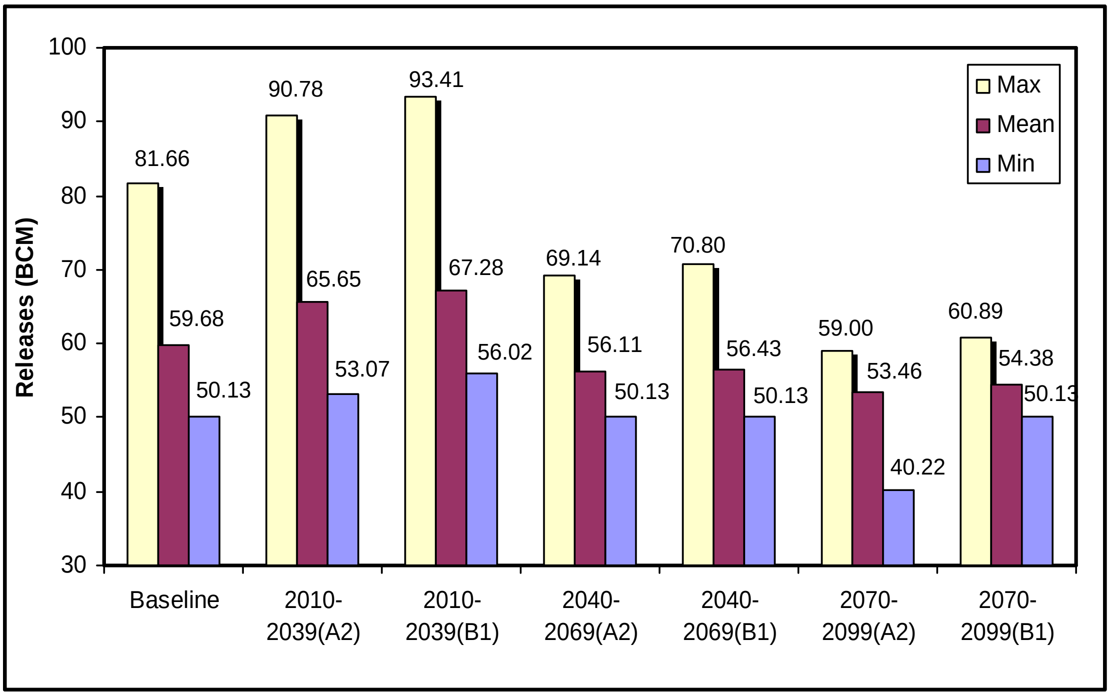


Figure 6.4(a). Annual withdrawal from the AHDR for scenario III.

6.2.3.2 Sensitivity of reservoir level variations to climate change

Figures A.3(d) and A.3(e) present the water level variation projected in the AHDR for baseline and three periods with two global emission scenarios A2 (B1). From these figures, it can be concluded that the maximum water levels upstream the AHD of all climate scenarios within the water year did not exceed 182 m, and the minimum water levels are above the minimum allowable limits (147 m). From this figures it can be noticed that the water levels limits and corresponding percentage of occurrence for scenario III are similar to scenario I, II for baseline and three periods with two global emission scenarios A2 (B1). Therefore, it is expected that the evaporation losses as well as the discharges to Toshka spillway in this scenario are very similar to the values in the two previous scenarios.

6.2.3.3 Sensitivity of hydropower production to climate change

Figure 6.4(b) shows the average annual hydropower production at the AHD for the A2 and B1 global emission scenarios. Under the baseline climate scenario, annual hydropower production at the AHD vary between 5972 - 11353 GWh, with a mean of about 8116 GWh. The annual average power production at the AHD generally follows changes in streamflow, increasing early in the century to 113 (117) percent of baseline production for the period I A2 (B1) emissions scenario, but then decreasing to 91 (92) and 80 (85) percent of the baseline mean for Periods II and III, respectively.

There are a few aspects of the frequency distributions that are also notable, maximum annual hydropower production occurs in period I (B1), and exceed 10000 GWh in approximately 35 % of years. For period III A2, the hydropower production stops for two months due to the reservoir levels falling below the minimum level for the hydropower generation, and the annual hydropower production is less than 8000 GWh in 83 % of years approximately (figure A.3(f)).

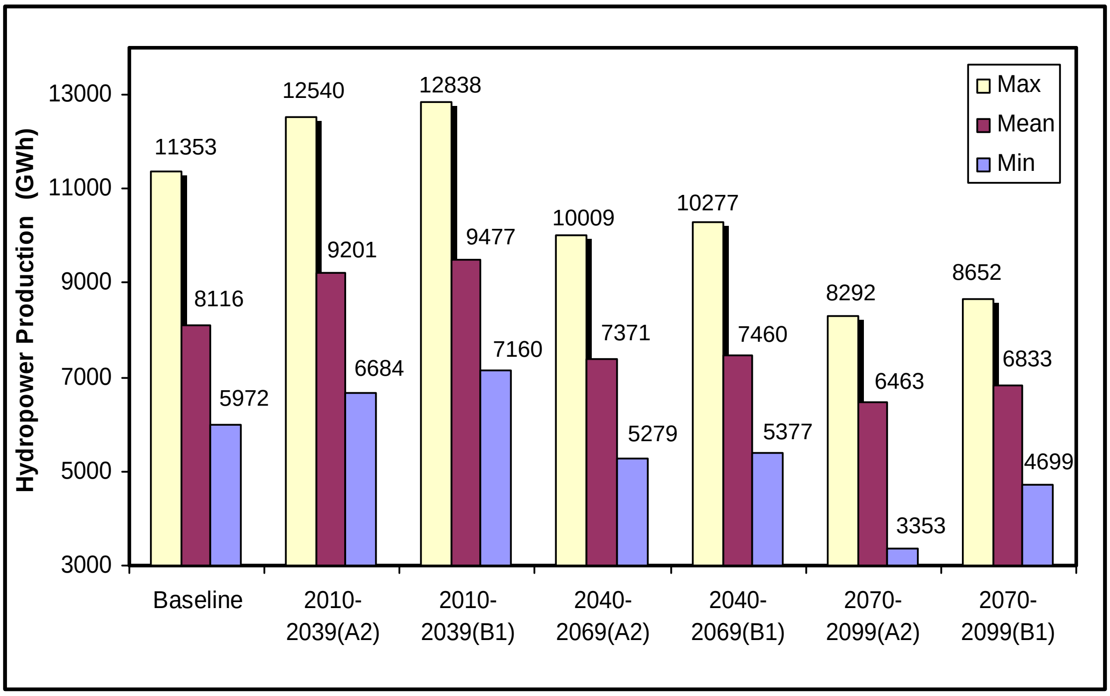


Figure 6.4(b). Annual hydropower production at the AHD for scenario III.

6.2.3.4 Sensitivity of evaporation losses to climate change

According to climate change scenarios, the annual evaporation losses vary between 8.76 – 13.26 BCM, with a mean of about 11.60 BCM for the baseline climate scenario. Due to streamflow increasing early in the century, the annual average evaporation losses increases to 12.36 (12.51) BCM for the period I A2 (B1) emissions scenario, but then decreases to 10.75 (10.89) and 9.12 (9.86) BCM for Periods II and III, respectively (figure 6.4(c)). Figure A.3(g) illustrates the frequency distribution of the annual evaporation losses, from this figure it can be noticed that 55 (59) and 27 (28) percent of years during periods I, II had evaporation losses greater than 12.5 BCB, compared to the baseline percent of 38 %, while in almost all years during period III had evaporation losses less than 12.5 BCB.

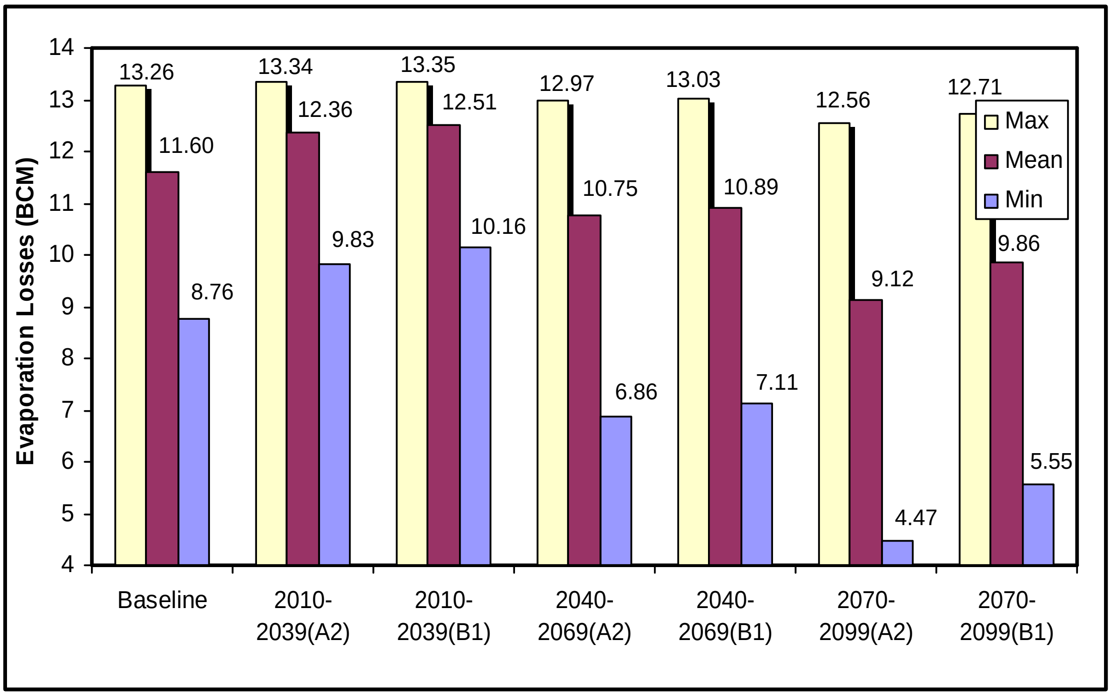


Figure 6.4(c). Annual evaporation losses for scenario III.

6.2.3.5 Sensitivity of Toshka spillway discharges to climate change

Under the baseline climate scenario, annual discharges to Toshka spillway vary between 0.67 – 9.20 BCB with a mean of about 3.00 BCM and occur in approximately 41 % of years (figure 6.4(d)). The outflows discharged to Toshka Spillway are negligible in period III due to reduction of the reservoir water levels in this period. For the period II, the range of projected discharges to Toshka spillway is 0.05 (0.10) to 4.91 (5.70) BCM and occur in approximately 38 (38) % of years for A2 (B1) emissions scenario (figure A.3(h)). The majority of scenarios show significant potential increasing in a mount of the released water to Toshka spillway for the period I, the range of projected releases grows to 0.07 (0.21)-11.53 (11.96) BCM and occur in approximately 69 (72) % of years for A2 (B1), respectively.

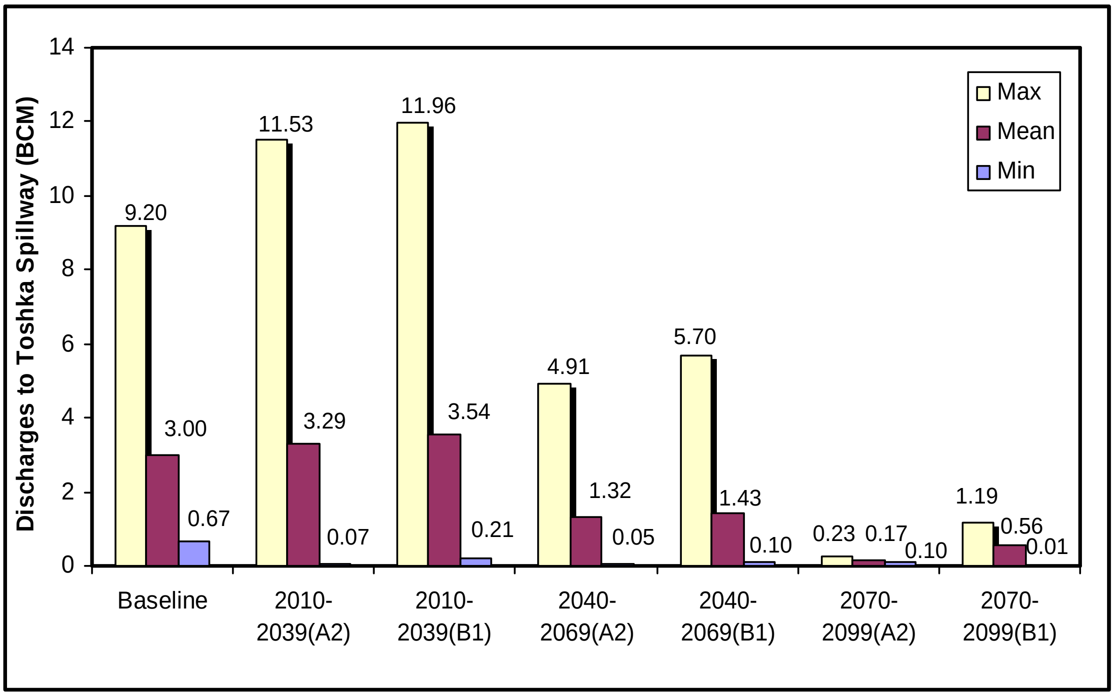


Figure 6.4(d). Annual Discharges to Toshka spillway for scenario III.

6.2.4 Development Scenario IV

In addition to the projects in previous scenarios, Scenario IV includes full implementation of "Baro-Akobo Multi-Purpose Water Resources Sub-Project" in Gambela, South-eastern Ethiopia on the Ethiopian-Sudanese border, and Egypt's annual fixed discharges are increased to 63 BCM.

6.2.4.1 Sensitivity of water supply releases to climate change

Figure 6.5(a) presents the water releases projected to occur if "Baro-Akobo Multi-Purpose Water Resources Sub-Project" is implemented. The curve shows improvements in the average annual releases from the AHDR, the mean annual withdrawal for the three periods (period I (2010-2039), period II (2040-2069), and period III (2070-2099)) and two global emission scenarios A2 (B1) are 69.90 (71.79), 59.81 (60.12) and 56.89 (57.92) BCM, respectively, compared to the baseline of 63.47 BCM

There are a few aspects of the frequency distributions (figure A.4(a)) that are also notable, the amount of minimum annual withdrawal from the AHDR increased from 37.8 BCM in scenario I during period III A2 to reaches to 43.07 BCM in scenario IV for the same period, and occurs in one year which presents approximately 3.40 % of years.

As a result of the serious reduction in the amount of water entering the reservoir, the withdrawal from the AHDR stops during period III A2 for two months.

In period I (B1), the maximum release downstream the AHD reaches to 4592 m^3/s (397 MCM/day) (figure A.4(b,c)). Also in this period, maximum spills over and above the fixed discharge program (63 BCM/year) occur in approximately 59 % of years, in 61 % of this cases the spills are greater than 10 BCM.

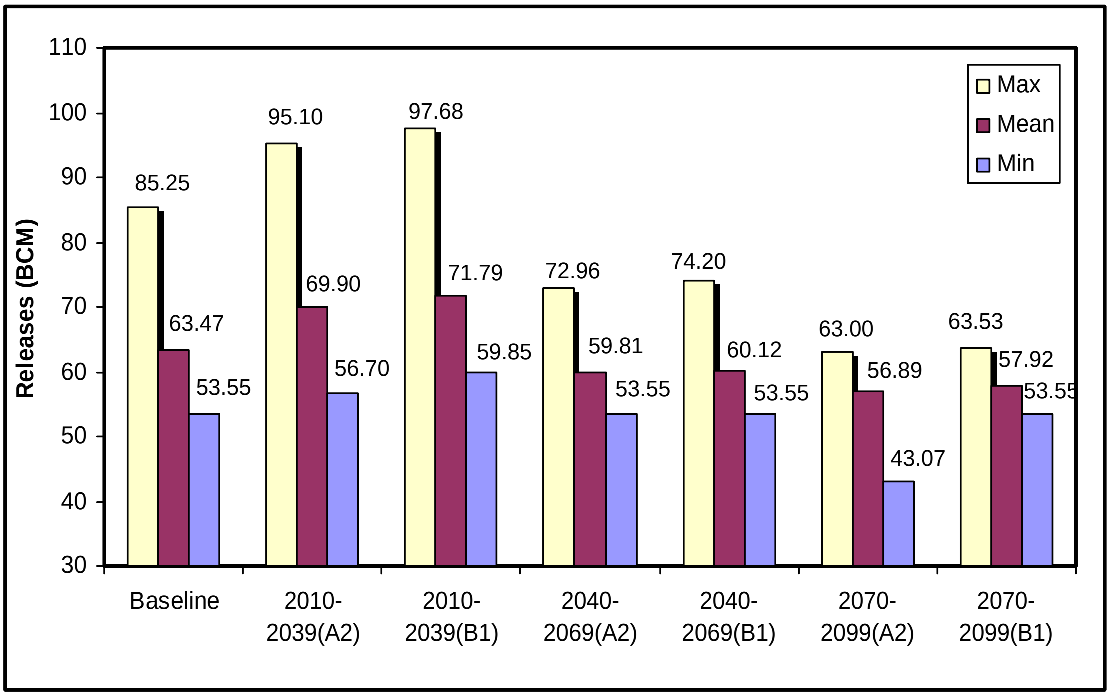


Figure 6.5(a). Annual withdrawal from the AHDR for scenario IV.

6.2.4.2 Sensitivity of reservoir level variations to climate change

It can be concluded from figure A.4(d) that the maximum water levels upstream the AHD of all climate scenarios within the water year did not exceed 182 m, and the minimum water levels are above the minimum allowable limits (147 m). Figure A.4(e) illustrates the frequency distribution of the levels in the reservoir, from this figure it can be noticed that the water levels limits and corresponding percentage of occurrence for scenario IV are similar to scenario I, II, and III for baseline and three periods with two global emission scenarios A2 (B1).

6.2.4.3 Sensitivity of hydropower production to climate change

Under the baseline climate scenario, annual hydropower production at the AHD vary between 6537 - 11903 GWh, with a mean of about 8722 GWh. The annual average power production at the AHD generally follows changes in streamflow, increasing early in the century to 113 (117) percent of baseline production for the period I A2 (B1) emissions scenario, but then decreasing to 91 (92) and 80 (85) percent of the baseline mean for Periods II and III, respectively (figure 6.5(b)).

For period III A2, the hydropower production stops for two months also due to the reservoir levels falling below the minimum level for the hydropower generation, and the annual hydropower production is less than 8000 GWh in more than 66 % of years. The maximum annual hydropower production occurs in period I (B1), and exceed 10000 GWh in approximately 45 % of years (figure A.4(f)).

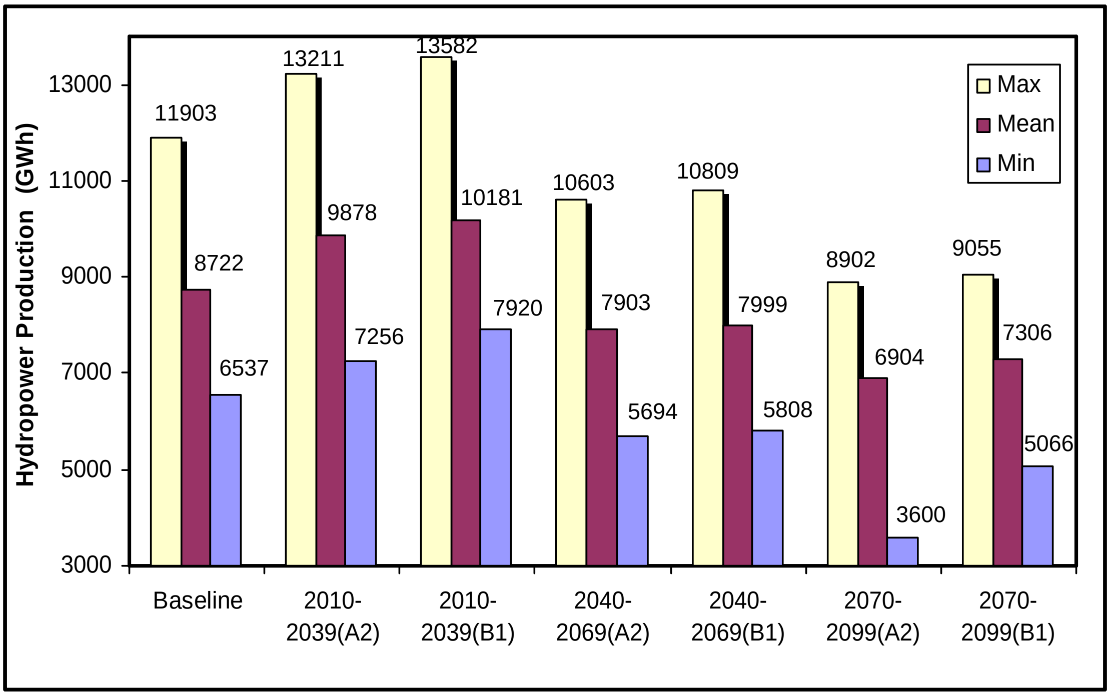


Figure 6.5(b). Annual hydropower production at the AHD for scenario IV.

6.2.4.4 Sensitivity of evaporation losses to climate change

Figure 6.5(c) shows the annual evaporation losses for the baseline and three periods with two global emission scenarios. Under the baseline climate scenario, the annual evaporation losses vary between 9.10 – 13.27 BCM, with a mean of about 11.71 BCM. The annual average evaporation losses increases to 12.44 (12.53) BCM for the period I A2 (B1) emissions scenario, but then decreases to 10.74 (10.90) and 9.07 (9.81) BCM for Periods II and III, respectively. It can be noticed from figure A.4(g) that 55 (62) and 24 (31) percent of years during periods I, II had evaporation losses greater than 12.5 BCB, compared to the baseline percent of 38 %, while in almost all years during period III had evaporation losses less than 12.5 BCB.

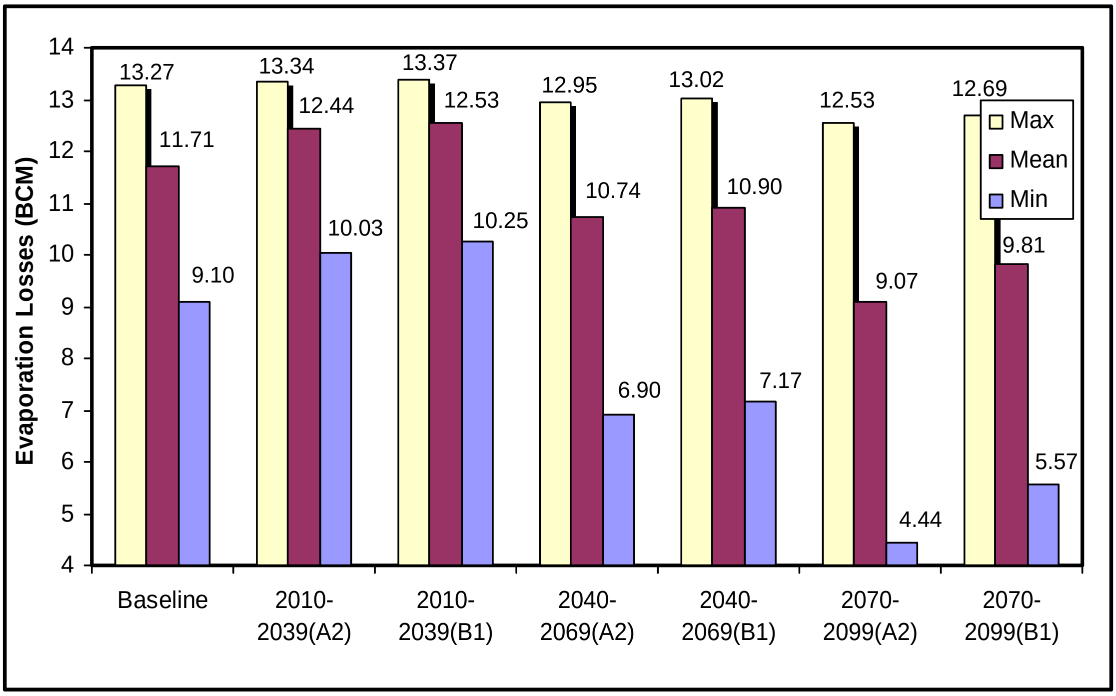


Figure 6.5(c). Annual evaporation losses for scenario IV.

6.2.4.5 Sensitivity of Toshka spillway discharges to climate change

Annual discharges to Toshka spillway vary between 0.60 – 9.29 BCB with a mean of about 2.98 BCM and occur in approximately 41 % of years for the baseline climate scenario (figure 6.5(d)). Discharges to Toshka spillway are negligible in period III due to reduction of the reservoir water levels in this period. For the period II, the range of projected discharges to Toshka spillway is 0.05 (0.05) to 4.62 (5.57) BCM and occur in approximately 34 (38) % of years for A2 (B1) emissions scenario (figure A.4(h)). The majority of scenarios show significant potential increasing in a mount of the released water to Toshka spillway for the period I, the range of projected releases to the spillway grows to 0.10 (0.03)-11.40 (12.18) BCM and occur in approximately 69 (76) % of years for A2 (B1), respectively.

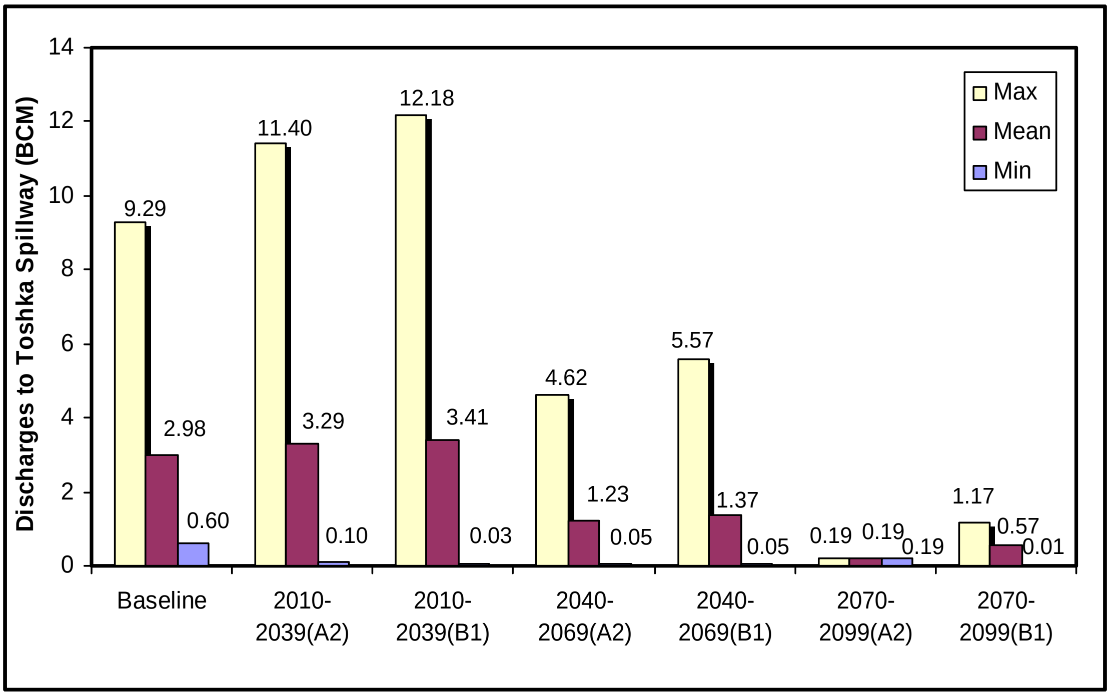


Figure 6.5(d). Annual Discharges to Toshka spillway for scenario IV.

7 ADAPTIVE OPERATION STRATEGIES

7.1 MODIFICATION OF THE OPERATION RULES

The aim of this chapter is to define the optimum adaptive reservoir operation strategies, which be able to cope with the potential future scenarios.
According to the existing operating rules for the AHDR, the present annual water releases from the reservoir is rather constant and not depending on the reservoir level. In order to improve the performance of the AHD and determine the optimal releases policy, it is advantageous to make reservoir releases dependent on the level and thus the volume in the reservoir. For this, a dynamic operation rule was devised which links the reservoir releases (Q_{out}) to the current reservoir level (H_i).
The release of the reservoir at any particular time is determined by its water level. Figure 7.1 depicts this rule. The rule divides the water levels into three zones: zone 1, zone 2, and zone 3. If the water level is in zone 2, the release equals a constant Q_o; if the water level is in zone 1, the release linearly slides to a minimum value; and if the water level is in zone 3, the release linearly increases to a maximum value. The boundaries of the zones, the constant value Q_o, and the minimum and the maximum values are all user specifiable [Rohde and Naparaxawong, 1981; Yao and Georgakakos, 2003; Loucks et al., 2005].

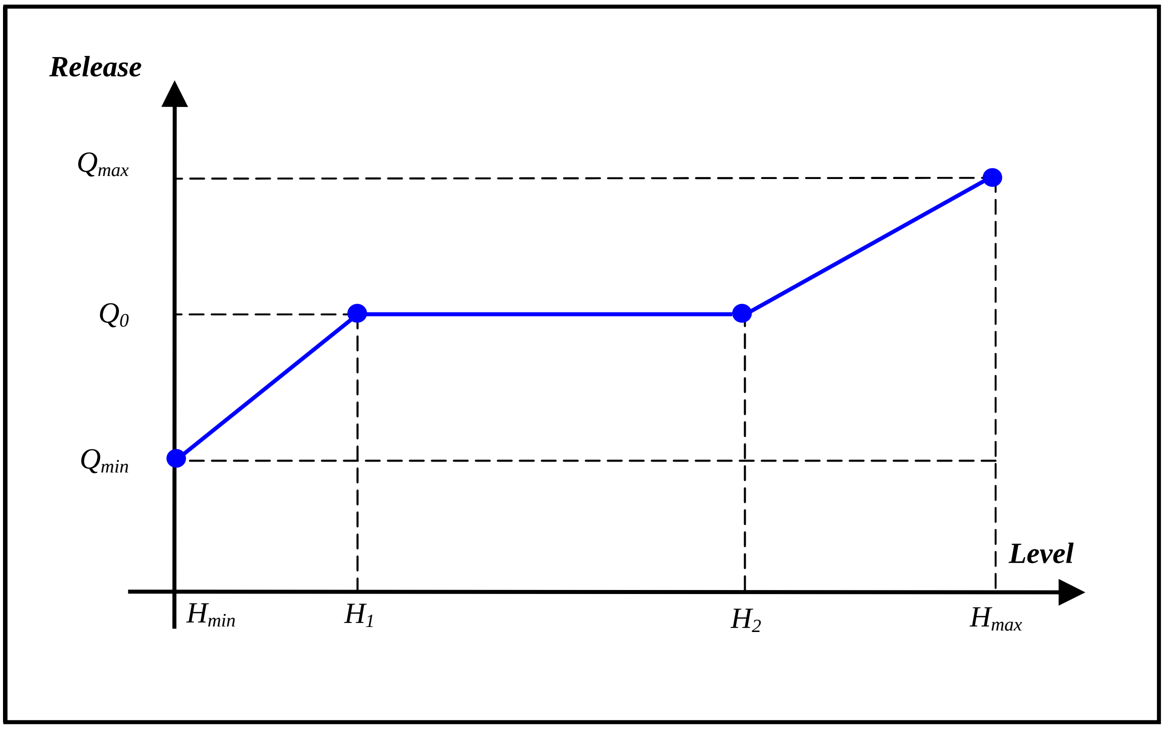


Figure 7.1. Release-Level rule curve.

The release is given by:

$$Q_{out} = \begin{cases} Q_{\min} + \dfrac{H_i - H_{\min}}{H_1 - H_{\min}}(Q_o - Q_{\min}) & H_i \le H_1 \\ Q_o & H_1 \le H_i \le H_2 \\ Q_o + \dfrac{H_i - H_2}{H_{\max} - H_2}(Q_{\max} - Q_o) & H_i \ge H_2 \end{cases}$$

7.2 OPTIMIZATION PROCESS

This study proposes to optimize the control strategies for the AHDR operation using the software BlueM, a simulation / optimization package has been developed for integrated river basin / reservoir systems.
The framework of the simulation-optimization process is shown in figure 7.2. First, different parameter sets defining the control strategies are generated. For each trial parameter set the simulation model is used to evaluate the performance of the system with respect to different objectives. Then, the parameter set is modified toward optimality by using evolutionary algorithms. The process is continued until one of the termination criteria is satisfied.

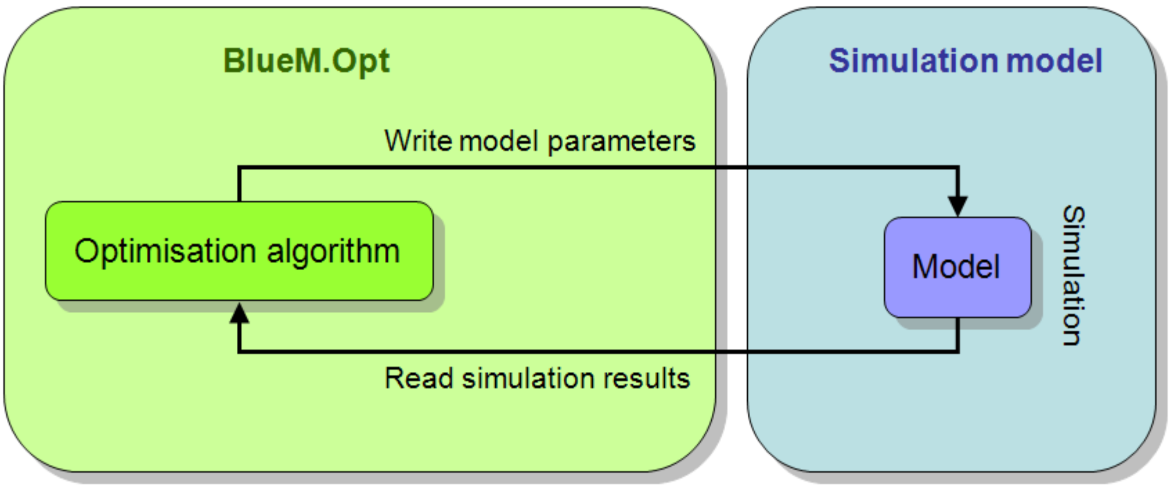


Figure 7.2. General framework of simulation-optimization modelling approach.

The common objectives that have been considered are:

- Minimize evaporation losses.
- Minimize discharges to Toshka spillway.
- Minimising downstream flood risk.
- Maximize water supply.
- Maximize the potential for hydropower generation.

7.3 A MULTI-OBJECTIVE OPTIMIZATION PROBLEM (MOP)

Solving an optimization problem is a complex task, the problem to be solved has multiple objectives conflicting across a highly multi-dimensional problem space. In addition, constraints as well as a multitude of degrees of freedom have to be considered. This kind of problem is often referred to as a multi-objective optimization problem (MOP).
The following definitions of the MOP are adopted from Deb (2001). According to Deb, a MOP has a number of objective functions which are to be minimized or maximized. The problem usually has a number of constraints which any feasible solution must satisfy. In the following, the multi-objective optimization problem is stated in its general form.

Minimize/Maximize	$f_m(x)$,	m=1, 2, ,…, M;
subject to	$g_j(x) \geq 0$,	j=1, 2, ,…, J;
	$h_k(x) = 0$	k=1, 2, ,…, K;
	$x_i^L \leq x_i \leq x_i^U$	i=1, 2, ,…, N;

A solution x is a vector of N decision variables ($x = x_1,\ldots, x_N$). The last set of constraints restricts each decision variable x_i ($i = 1,\ldots, N$) to take a value within a lower and an upper bound. These bounds constitute a decision variable space Ω. Associated with the problem are J inequality and K equality constraints. A solution x that does not satisfy all of the J+K constraints and all of the 2N variable bounds stated above is called an infeasible solution. On the other hand, if a solution x satisfies all constraints and variable bounds, it is called a feasible solution.
In the presence of constraints, it is not necessary that the entire decision variable space Ω is feasible. In multi-objective optimization, the objective functions constitute a multi-dimensional space, in addition to the usual decision variable space. This additional space is called the objective function space Λ. Each N-dimensional solution vector x in the decision variable space, is mapped onto an M-dimensional objective vector in the objective space.
A solution x is termed Pareto optimal when there is no feasible solution x' that will improve at least one objective function value without worsening at least one other objective function value. The Pareto set is the set of Pareto optimal solutions, which is also called the set of non-dominated or non-inferior solutions. The Pareto front is the mapping of the Pareto set from the decision variable space onto the objective function space.
In order to solve multi-objective optimization problems, an appropriate optimization algorithm needs to be chosen. Evolutionary Algorithms (EAs) belong to the global optimization procedures, which are designed for locating the global optimum whilst not getting stuck in local optima. EAs is an umbrella term for a number of different optimization methodologies which are based on similar fundamental concepts. These algorithms use mathematical abstractions of the evolution procedure for the search of optimum solutions. Since the 1970s, several methodologies have been proposed, mainly genetic algorithms, evolutionary programming, and evolution strategies [Baeck et al., 1997].

All these algorithms use the processes of mutation and recombination to modify decision variables (parameter set). Subsequently, the performance is checked with regard to quality criteria. Following the principle "survival of the fittest" only the "strongest" samples will survive and generate the next generation based on these processes. This class of algorithms does not make assumptions about the continuity of the objective function and does not require information on its derivatives.
In addition, EAs allow for the consideration of linear and non-linear constraints and the handling of complex optimization problems. EAs are therefore appropriate optimization methods where simulation-based evaluation of the objectives is required like in the case of the AHDR.
Classical approaches to multi-objective optimization, including simple Evolutionary Algorithms, convert a MOP into a single objective optimization problem (e.g. by a weighting sum approach). With this approach, only one Pareto optimal solution can be found in each optimization run. If more than one solution is necessary, a repeated application is required. In addition, not all solutions will necessarily be found (especially in the case of non-convex optimization problems) and subjective information is needed (e.g. assumption of problem-specific weighting factors). However, due to their simplicity and ease of implementation, these approaches are widely used in solving MOP. Most EAs use a population of solutions in each iteration instead of a single solution. Therefore, the result of an Evolutionary Algorithm is also a population of solutions. If an optimization problem has multiple optimal solutions, Evolutionary Algorithms can be used to capture multiple optimal solutions in its final population. Hence, EAs are particularly suited for multi-objective optimization since they allow the determination of the Pareto set of the solutions and the consideration of linear and non-linear constraints in a single optimization run [Deb, 2001].
In addition, they offer a less subjective means of finding multiple solutions because they need only few or no problem specific information. Depending on the preferences of a decision-maker, the remaining task is to choose a group of solutions (if necessary for a more detailed analysis) [Farmani et al., 2006].

7.4 OPTIMAL OPERATION RULES

The Multi-Objective Evolution Strategy was applied to the problem in order to determine the optimal release policy with respect to evaporation losses, discharges to Toshka spillway, flood protection, water supply security and hydropower production. Figure 7.3 shows sample of optimization, the final coordinates for the release-level function for the different scenarios are therefore set as shown in figure 7.4.

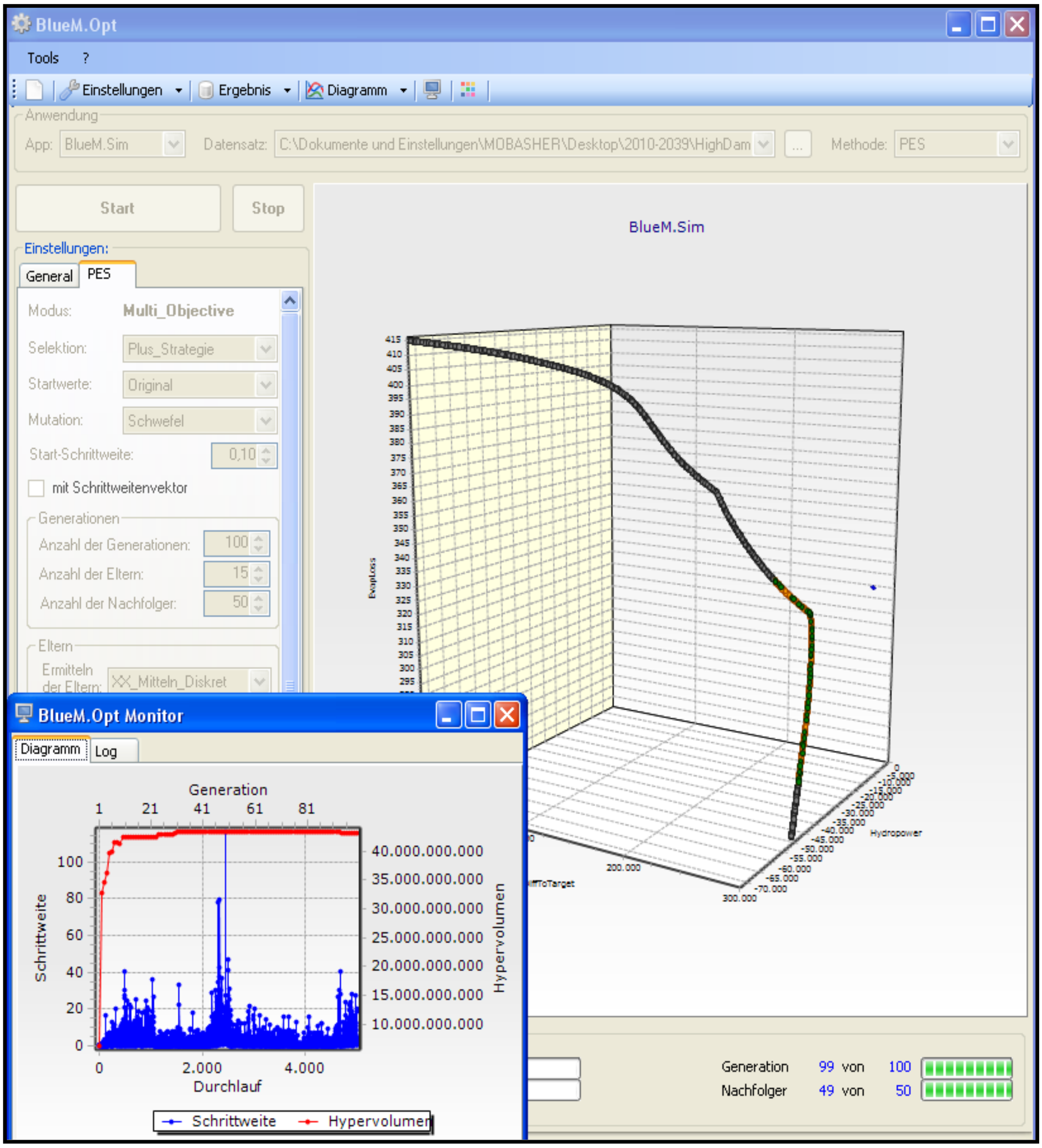


Figure 7.3. Sample of optimization results from software "BlueM.Opt".

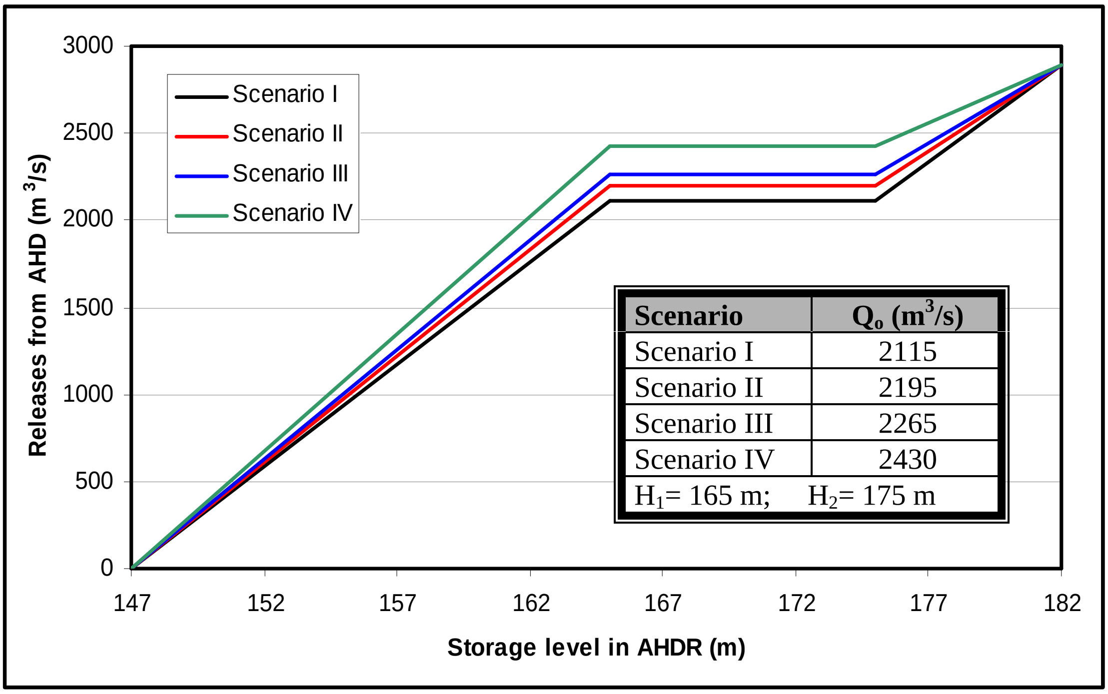


Figure 7.4. Final release-level-relation of the optimal operation rule for different scenarios.

7.5 EVALUATION OF THE OPTIMAL OPERATION RULE

In order to evaluate the optimal release policy, the following sections present comparison between the current reservoir operation rules (CUR) which were determined by the Ministry of Water Resources and irrigation, and developed optimal operation rule (OPT). More detailed information on the entire cumulative frequency of the results is provided and discussed in Appendix B.

7.5.1 Evaporation Losses

First, the primary objective of the OPT is to reduce the evaporation losses from the reservoir and thus increase water availability from the AHDR. According to the OPT policy, the minimum scenario of the annual evaporation losses occurs in period III A2 with an average range varies from 5.81 – 11.69 BCM with a mean of about 7.30 BCM, compared to an average range varies from 4.53 – 12.58 BCM with a mean of about 9.21 BCM for the CUR policy at the same period. The maximum scenario of the annual evaporation losses occurs in period I (B1) with an average range varies from 6.75 – 12.87 BCM with a mean of about 10.05 BCM, compared to an average range varies from 10.26 – 13.37 BCM with a mean of about 12.54 BCM for the CUR policy. Figure 7.5 illustrates the frequency distribution of maximum and minimum annual evaporation losses, from this figure it can be noticed that for the CUR policy 72 percent of years during maximum scenario had evaporation losses greater than 12.5 BCB, this percentage decreases to 16 percent of years for the OPT policy. In minimum scenario, 45 percent of years had evaporation losses greater than 10.00 BCB, while in almost all years for the OPT policy had evaporation losses less than 10.00 BCB.

Generally, it can be concluded from this comparison (see also table B.1), there is a possibility to reduce the evaporation losses through applying the OPT policy. This policy would save an estimated average annual amount of some 2.70 BCM of evaporation losses from the AHDR

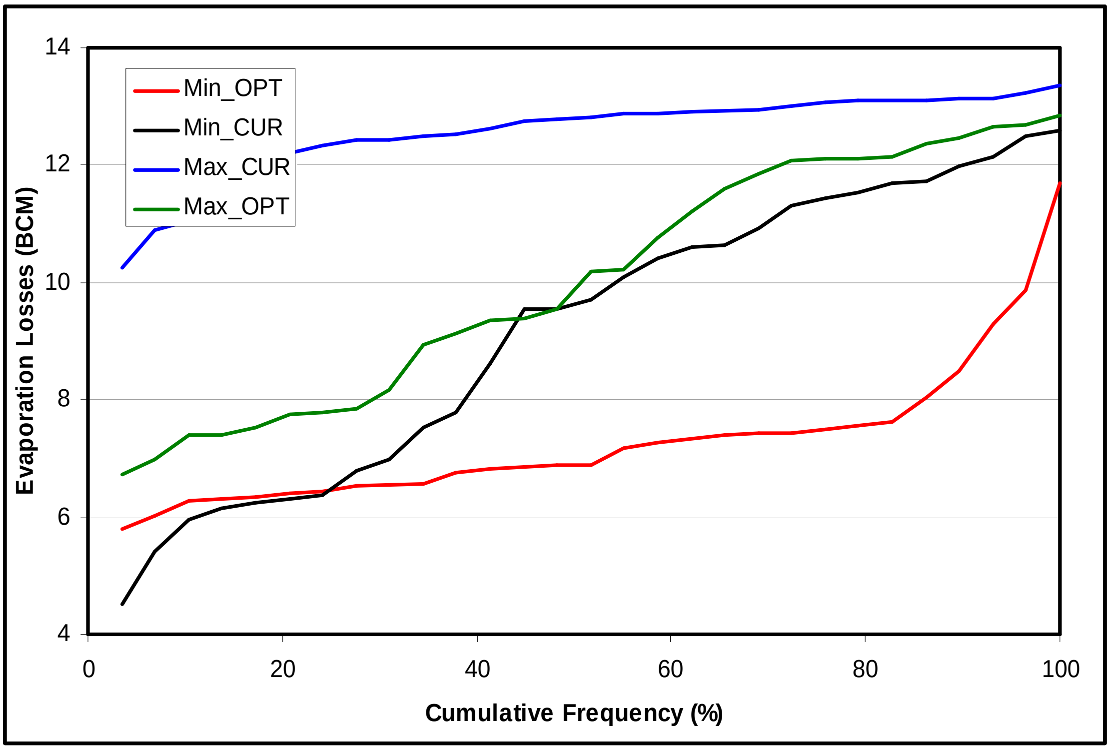


Figure 7.5. Comparison of frequency distribution curves of minimum and maximum annual evaporation losses scenario as produced by optimal operation rule (OPT) and current operation rule (CUR).

7.5.2 Discharges to Toshka Spillway

Due to the reduction in the reservoir levels as a result of implementation of the OPT policy (see Appendix B), and as a consequence the evaporation loses could be lowered. In addition, the average range of discharges to Toshka spillway in maximum scenario (period I (B1)) reduced significantly from 0.12 – 12.50 BCB with a mean of about 3.52 BCM and occur in approximately 69 % of years for the CUR policy, to 0.02 – 4.52 BCB with a mean of about 1.26 BCM and occur merely in 21 % of years for the OPT policy (figure 7.6). If the OPT had been followed, and if the objective was simply reduce discharges to Toshka spillway, it would never have been necessary to use Toshka spillway during the periods I and II for all scenarios. Moreover in this policy, the reservoir level not exceeds 182 meters and thus there is still a certain margin of safety.

In addition, via reduction of spillage to Toshka, there is a possibility to water saving in the order of 1.60 BCM/year through a change in reservoir operation from the CUR to the OPT policy.

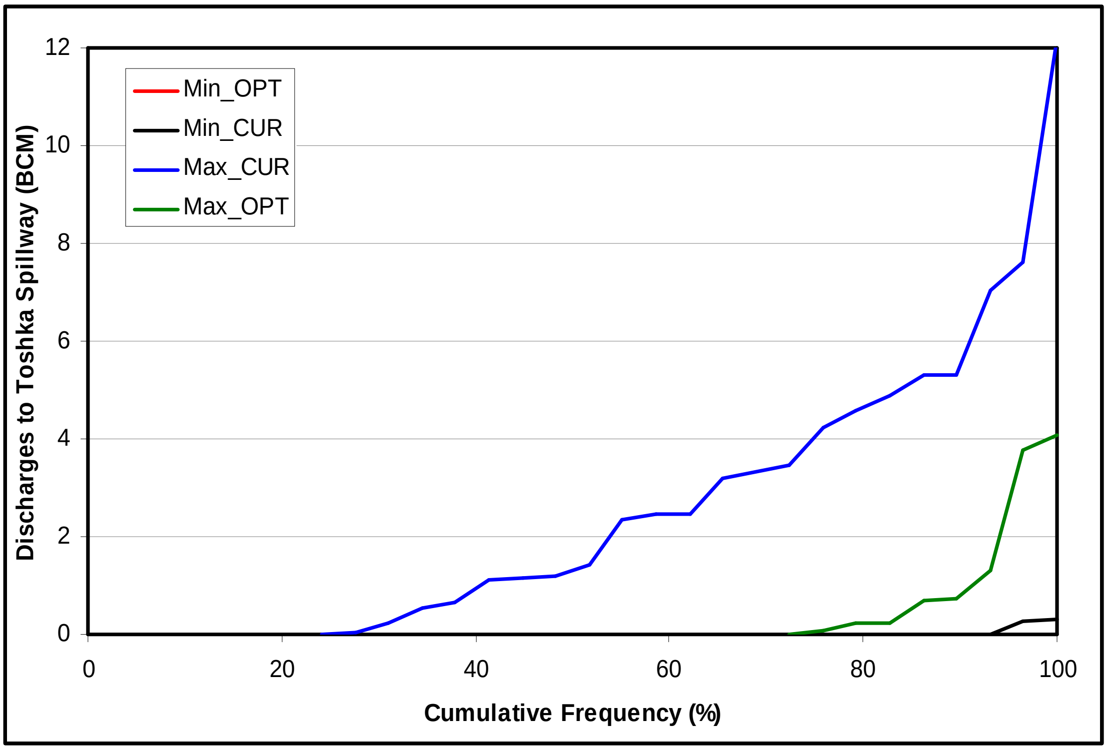


Figure 7.6. Comparison of frequency distribution curves of minimum and maximum scenario of annual discharges to Toshka spillway as produced by optimal operation rule (OPT) compared to current operation rule (CUR).

7.5.3 Downstream Flood Risk

One of the most important objective is the avoidance of excessive (degradation causing) releases. Of particular interest are both the frequencies and the magnitudes of these releases. From figure 7.7 it can be seen that the CUR policy not only has the most excessive releases, but also the most severe. The OPT policy reduces both the frequency and the magnitude. For example, during period I (B1) in scenario IV the maximum releases downstream the AHD for the OPT policy was 2861 m^3/s compared to 4592 m^3/s for the CUR policy at the same period.

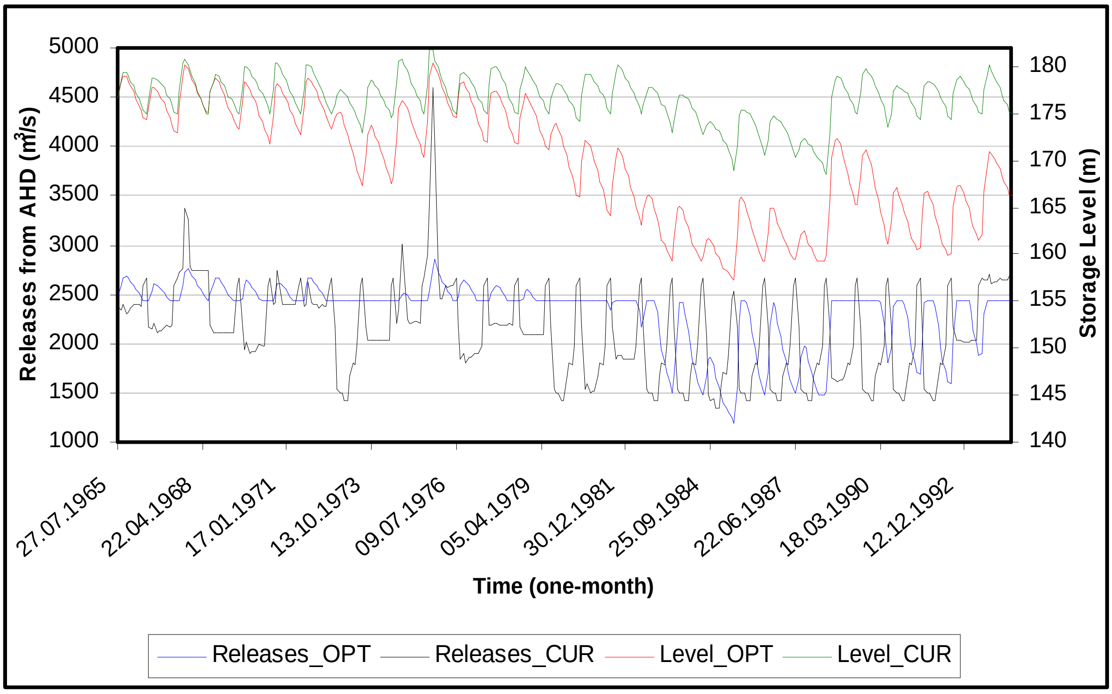


Figure 7.7. Max. reservoir storage levels and releases from the AHD as produced by optimal operation rule (OPT) compared to current operation rule (CUR).

7.5.4 Water Supply Releases

Conserving water to ensure meeting present and potential future downstream demands scenarios is one of the major benefits for the OPT policy. Figure 7.8 gives comparison of frequency distribution curves of minimum and maximum annual withdrawal scenario from the reservoir as produced by the OPT policy and the CUR policy. From figure 7.8, it can be seen that for the OPT policy, Periods of failure are less frequently and of shorter duration than for the CUR policy. For example, during period III A2 in scenario I which represents the minimum scenario, the mean annual withdrawal from the AHDR increased from 50.42 BCM with the CUR policy to 52.68 BCM with the OPT policy, and the percentage of short falls of Egypt's target demand in this period decreases also from approximately 62 % of years for the CUR policy to 55 % of years for the OPT policy. At the same period there was a significant probability for the CUR policy, the withdrawal from the AHDR stopped for two months, while for the OPT policy there is no interruption of the withdrawal from the reservoir.
On the other hand, the mean annual withdrawal from the AHDR during period I (B1) in scenario IV which represents the maximum scenario increased to 78.02 BCM for the OPT policy instead 71.79 BCM for the CUR policy (more details in table B.3).

On the whole, the comparison between the OPT policy and the CUR policy for each scenarios shows possible water saving in order of 5 BCM/year approximately throw a change in reservoir operation.

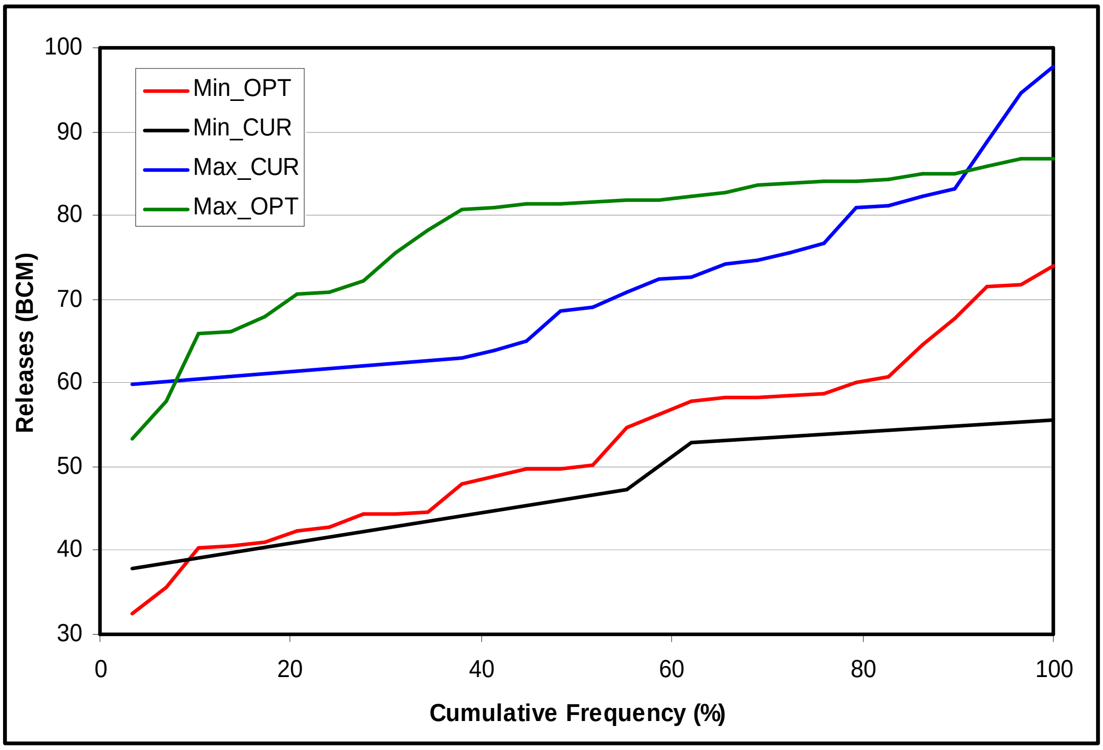


Figure 7.8. Comparison of frequency distribution curves of minimum and maximum annual withdrawal scenario from the reservoir as produced by optimal operation rule (OPT) and current operation rule (CUR).

7.5.5 Hydropower Production

The fifth objective concerns attaining yearly targets of hydropower generation. Figure 7.9 presents comparison of frequency distribution curves of minimum and maximum annual hydropower production scenario from the AHD as produced by the OPT policy and the CUR policy. From this figure (and more details in table B.4), it can be seen that, although the average reservoir levels could be reduced when using OPT policy, the annual average hydropower production at the AHD under the OPT policy increases to 102 percent of average hydropower production under the CUR policy for the period I in almost of scenarios, but then decreases to 95 percent of average hydropower production under the CUR policy for Periods II and III.

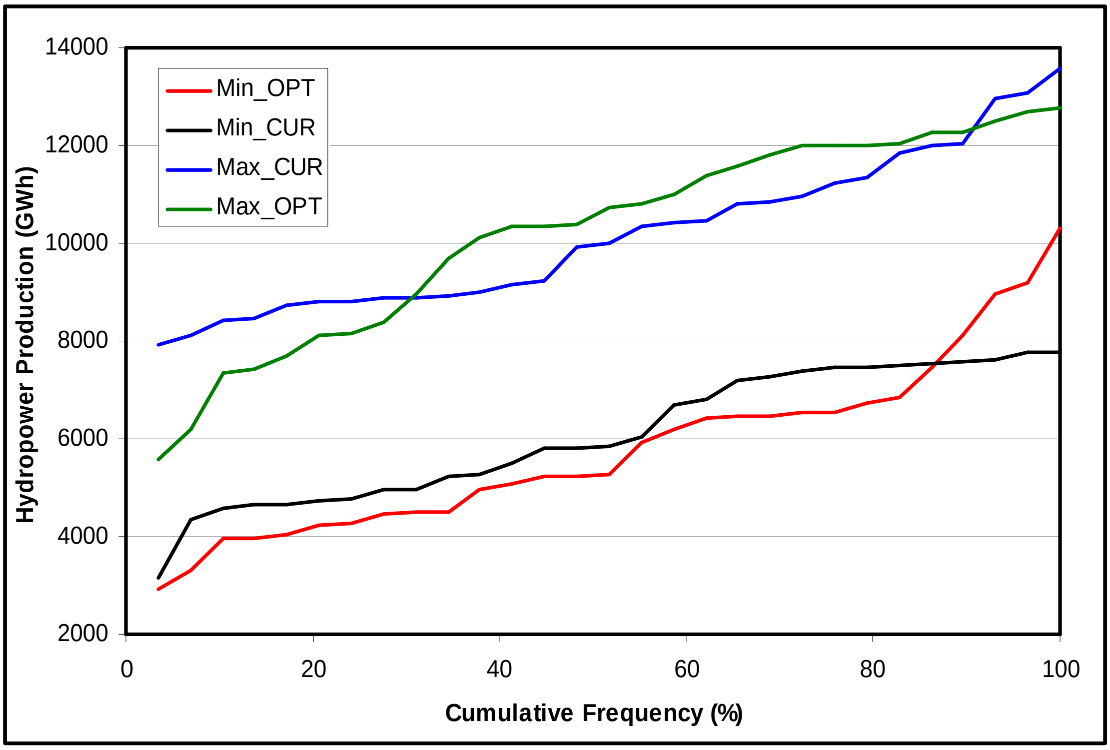


Figure 7.9. Comparison of frequency distribution curves of minimum and maximum annual hydropower production scenario from AHD as produced by optimal operation rule (OPT) and current operation rule (CUR).

8 VARIATION OF THE AHDR WATER LEVELS DERIVED FROM SATELLITE ALTIMETRY

8.1 INTRODUCTION

Concerning surface waters (rivers, lakes, man-made reservoirs, wetlands and inundated areas), in-situ gauging networks have been installed for several decades at least in some hydrographic basins. In situ measurements provide time series of water levels and discharge rates, which are used for studies of regional climate variability as well as for socio-economic applications (e.g., water resources allocation, navigation, land use, hydroelectric energy, flood hazards). Gauging stations, however, are scarce or even absent in parts of large river basins due to geographical, political or economic limitations. Moreover, since the beginning of the 1990s, numerous in-situ networks have declined or stopped working, because of political and economic factors.
Recently, remote sensing techniques have been used to monitor components of the water balance of large river basins on time scales ranging from months to decades. Among these, two are particularly promising: satellite altimetry for systematic monitoring of water levels of large rivers, lakes and floodplains and the new space GRACE gravity mission for measurement of spatio-temporal variations of land water storage. Other remote sensing techniques, such as Synthetic Aperture Radar (SAR) Interferometry and passive and active microwave observations also offer important information on land surface waters, such as changing areal extent of large wetlands.
By complementing in situ observations and hydrological modelling, space observations have the potential to improve significantly our understanding of hydrological processes at work in large river basins and their influence on climate variability, geodynamics and socio-economic life. Unprecedented information can be expected by combining models and surface observations with observations from space, which offer global geographical coverage, good spatio-temporal sampling, continuous monitoring with time, and capability of measuring water mass change occurring at or below the surface [LEGOS, 2009].

8.2 SATELLITE ALTIMETRY

Radar altimetry from space consists of vertical range measurements between the satellite and water level. Difference between the satellite altitude above a reference surface (usually a conventional ellipsoid), determined through precise orbit computation, and satellite-water surface distance, provides measurements of water level above the reference (figure 8.1). Placed onto a repeat orbit, the altimeter satellite over-flies a given region at regular time intervals (called the orbital cycle), during which a complete coverage of the Earth is performed [LEGOS, 2009].

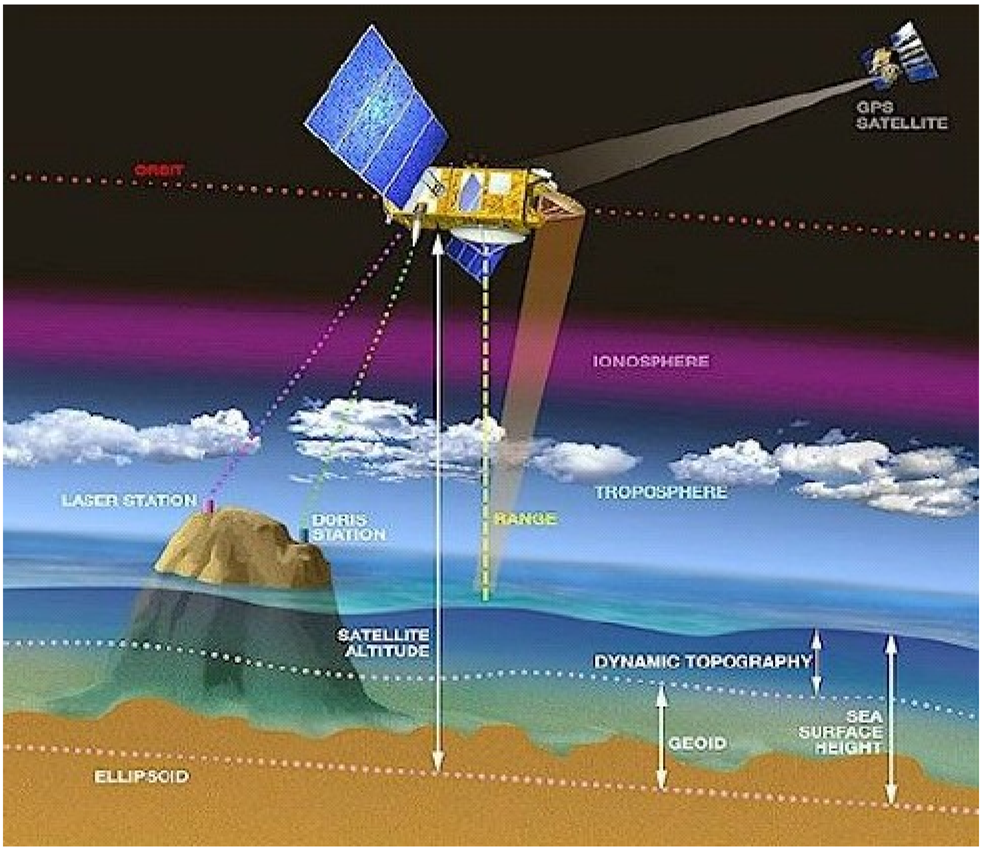


Figure 8.1. Working principle of sea (or lakes) level measurements.

8.3 SURFACE WATERS MONITORING BY SATELLITE ALTIMETRY

Water level measurement by satellite altimetry has been developed and optimized for open oceans. Nevertheless, the technique is now applied to obtain water levels of inland seas, lakes, rivers, floodplains and wetlands. Several satellite altimetry missions have been launched since the early 1990s: ERS-1 (1991-1996), Topex/Poseidon (1992-2006), ERS-2 (1995-), GFO (2000-), Jason-1 (2001-) and ENVISAT (2002-). ERS-1, ERS-2 and ENVISAT have a 35-day temporal resolution (duration of the orbital cycle) and 80 km inter-track spacing at the equator. Topex/Poseidon and Jason-1 have a 10-day orbital cycle and 350 km equatorial inter-track spacing. GFO has a 17-day orbital cycle and 170 km equatorial intertrack spacing. The combined global altimetry data set has more than decade-long history and is intended to be continuously updated in the coming decade. Combining altimetry data from several in-orbit altimetry missions increases the spatio-temporal resolution of the sensed hydrological variables [LEGOS, 2009].

8.4 WATER LEVEL DATA FOR THE AHDR

Resevoir level data and images for this analyses were obtained from internet website for LEGOS (Laboratorie d'Etudes en Geophysique et Oceanographie Spatiales (France)).
Figure 8.2 shows the Satellite altimetry missions tracks over the AHDR, the lake levels are based on merged Topex/Poseidon, Jason, ENVISAT and GFO data provided by ESA, NASA and CNES data centers. The altimeter range measurements used for lakes consist of 1Hz data. All classical corrections (orbit, ionospheric and tropospheric corrections, polar and solid Earth tides and sea state bias) are applied. Depending on the size of the lake, the satellite data may be averaged over very long distances. It is thus necessary to correct for the slope of the geoid (or equivalently, the mean lake level). Because the reference geoid provided with the altimetry measurements (e.g., EGM96 for T/P data) may not be accurate enough, a mean lake level have been computed, averaging over time the altimetry measurements themselves.
The water levels are further referred to this ‘mean lake level’. If different satellites cover the same lake, the lake level is computed in a 3-step process. Each satellite data are processed independently. Potential radar instrument biases between different satellites are removed using T/P (Topex/Poseidon) data as reference. Then lake levels from the different satellites are merged on a monthly basis (recall that the orbital cycles vary from 10 days for T/P and Jason, to 17 days for GFO and 35 days for ERS and Envisat) [LEGOS, 2009].

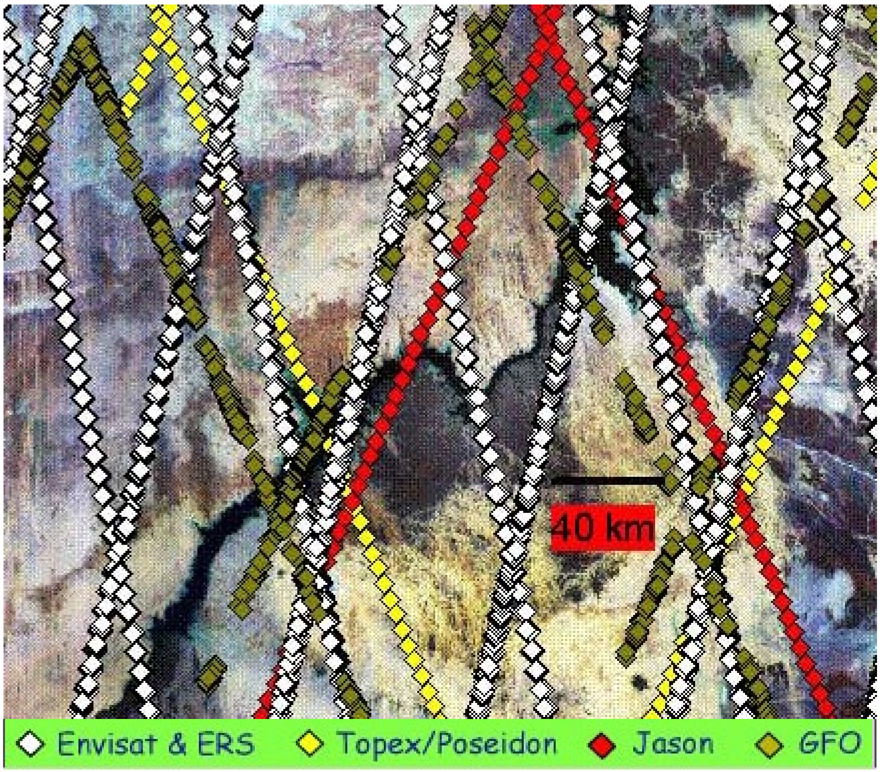


Figure 8.2. Satellite altimetry missions tracks over the AHDR.

8.5 RESERVOIR LEVEL COMPARISON

In principle, monitoring reservoir level variations, using satellite altimetry, is straightforward. Based on such a rationale, datasets from 10 tracks of ENVISAT missions for 8 years time span (2002-2009) are used, this is because ENVISAT data sets are available since 2002 to 2009. The location of ENVISAT missions over the AHDR are shown in figure 8.3, and all of the available data about the water level in the reservoir at the certain tracks are presented with more details in appendix C.
The altimeter-derived relative reservoir level height variations are evaluated as a function of time, figures 8.4 through 8.9 show the results of the comparison between reservoir level for some of satellite tracks over the AHDR. The results indicates to a general decline in the water surface in the reservoir, the average level variation between tracks Env_Nil_227_06 (90 km upstream the AHD) and Env_Nil_227_03 (49 km upstream the AHD) is about 0.27 m, this variation reaches to 0.35 m for tracks Env_Nil_872_03 (104 km upstream the AHD) and Env_Nil_872_01 (23 km upstream the AHD). In order to estimate the average level variation between the level in the reservoir at Toshka spillway and the level at the AHD, figure 8.9 represents reservoir levels variation between tracks Env_Nil_414_04 (upstream Toshka spillway) and Env_Nil_872_01(23 km upstream the AHD). It can be concluded from figure 8.9 that there is a level variation between the reservoir level at Toshk spillway and at the AHD some about 0.43 m in average.

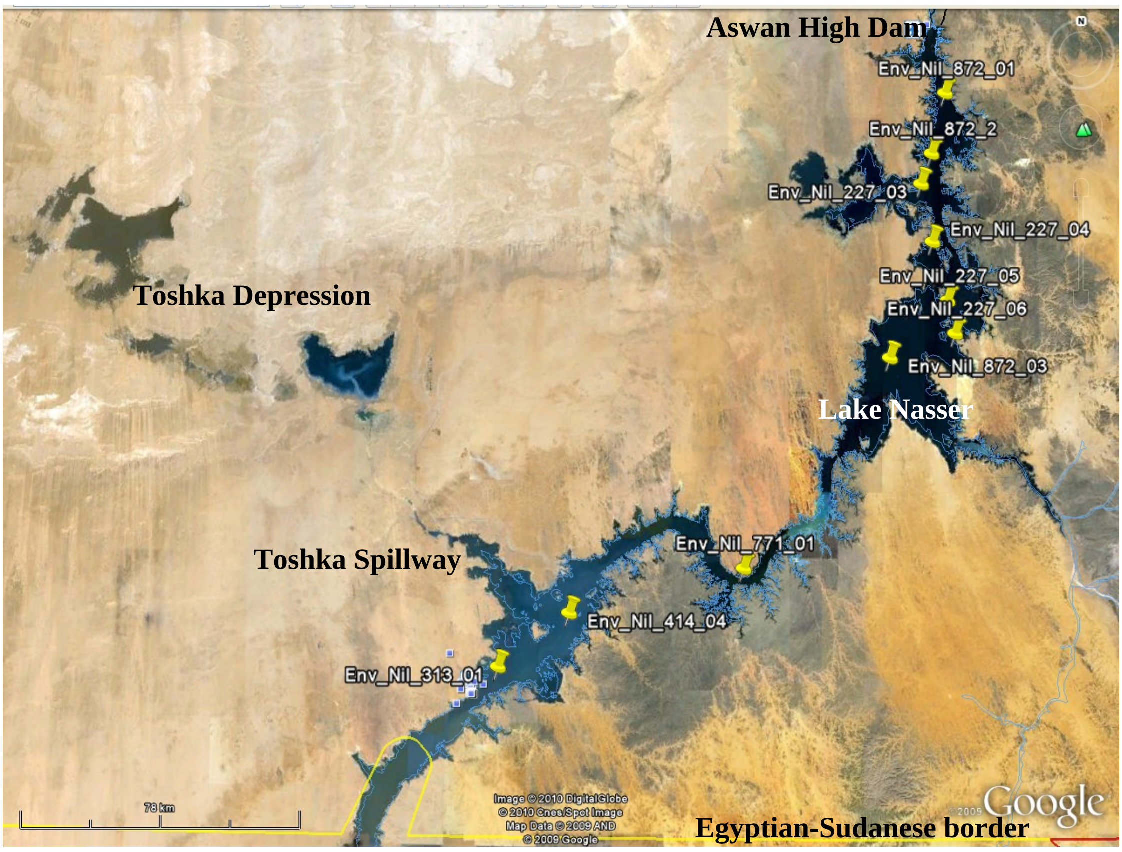


Figure 8.3. Satellite tracks location over the AHDR [Using google earth].

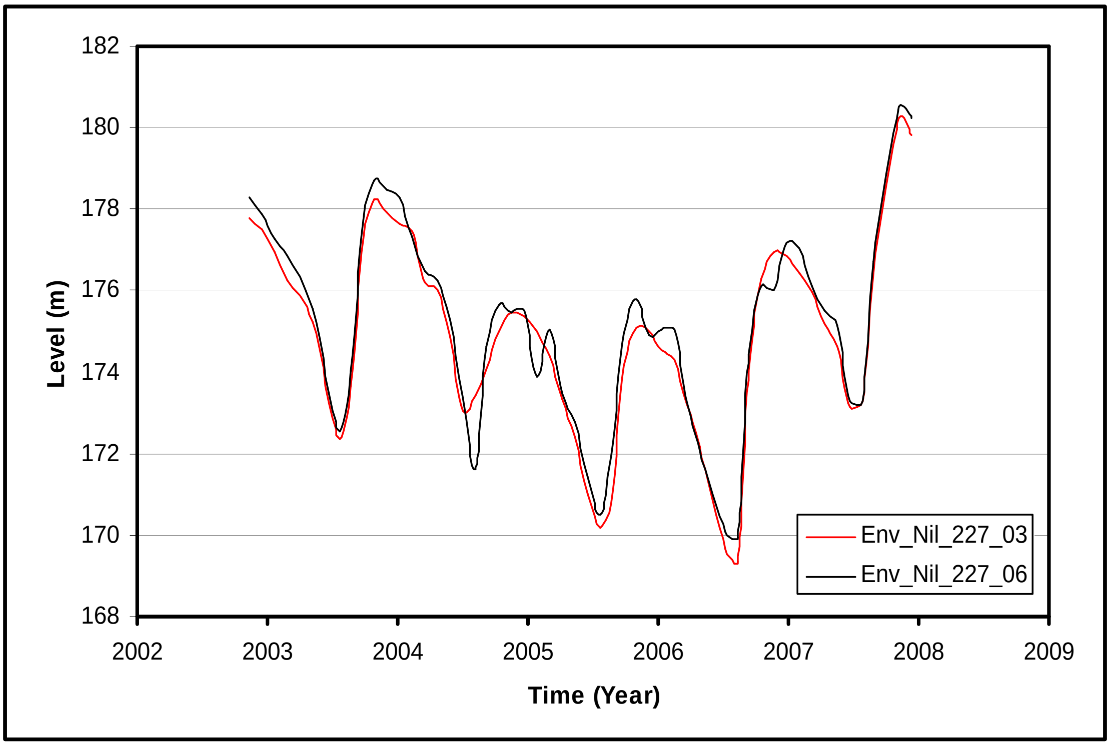


Figure 8.4. Reservoir levels variation between tracks Env_Nil_227_06 & Env_Nil_227_03.

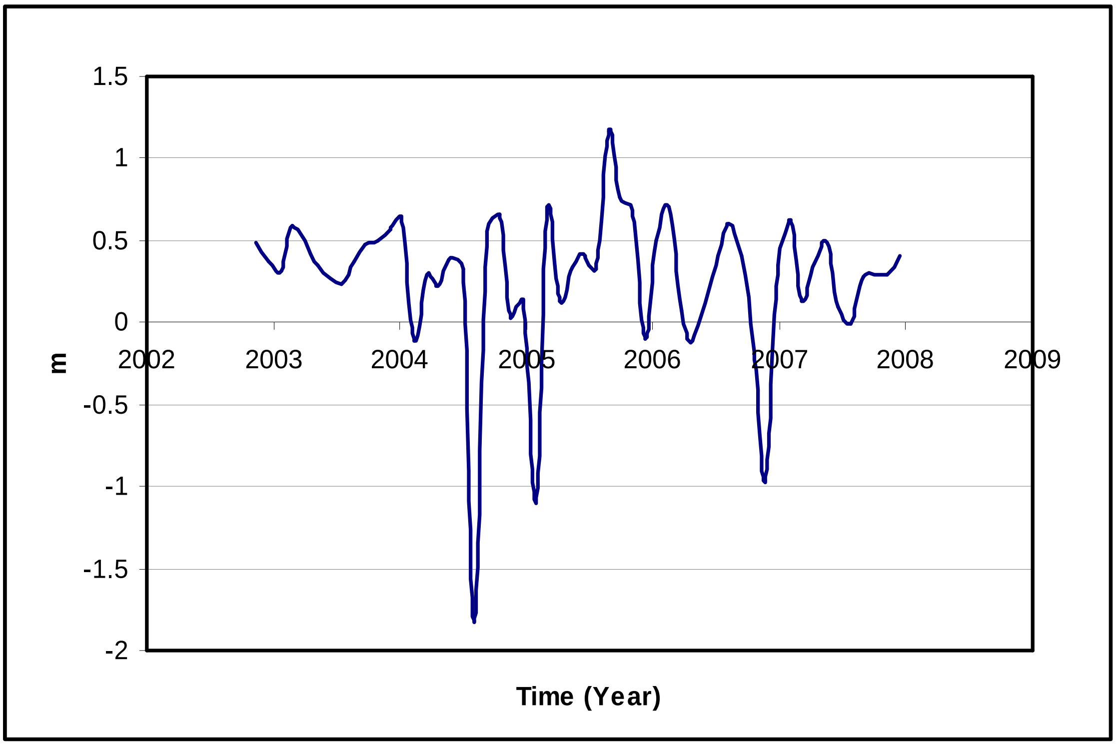


Figure 8.5. Reservoir levels variation between tracks Env_Nil_227_06 & Env_Nil_227_03.

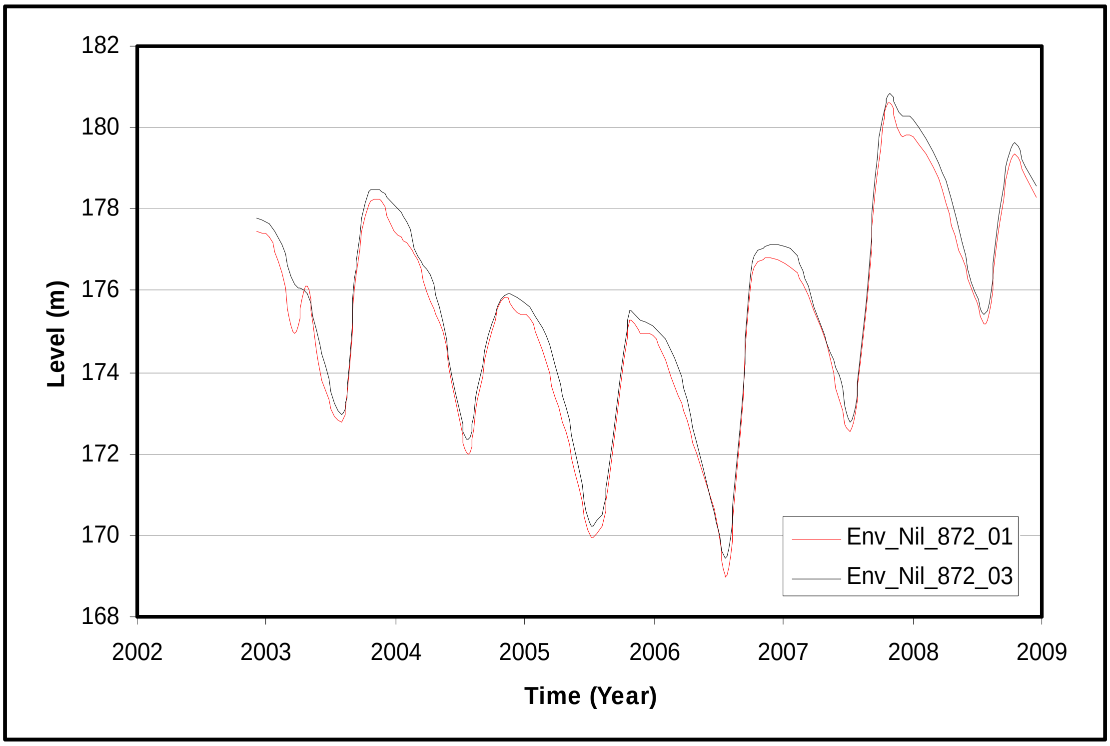


Figure 8.6. Comparison between the reservoir levels at tracks Env_Nil_872_03& Env_Nil_872_01.

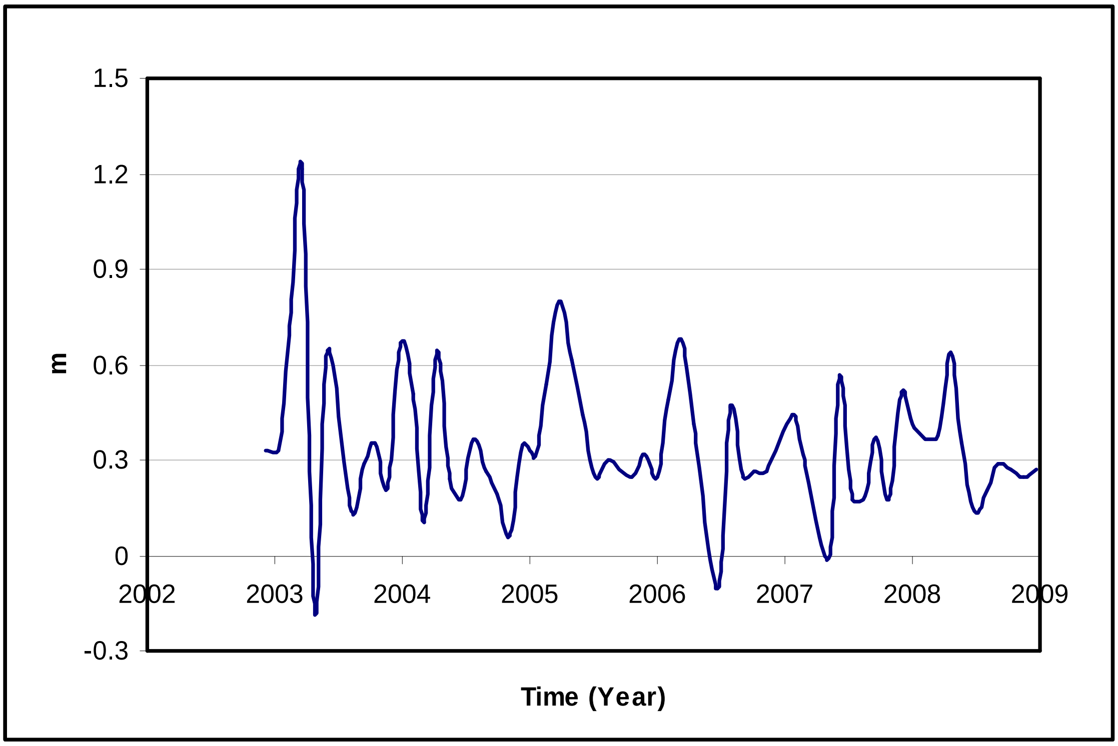


Figure 8.7. Reservoir levels variation between tracks Env_Nil_872_03& Env_Nil_872_01.

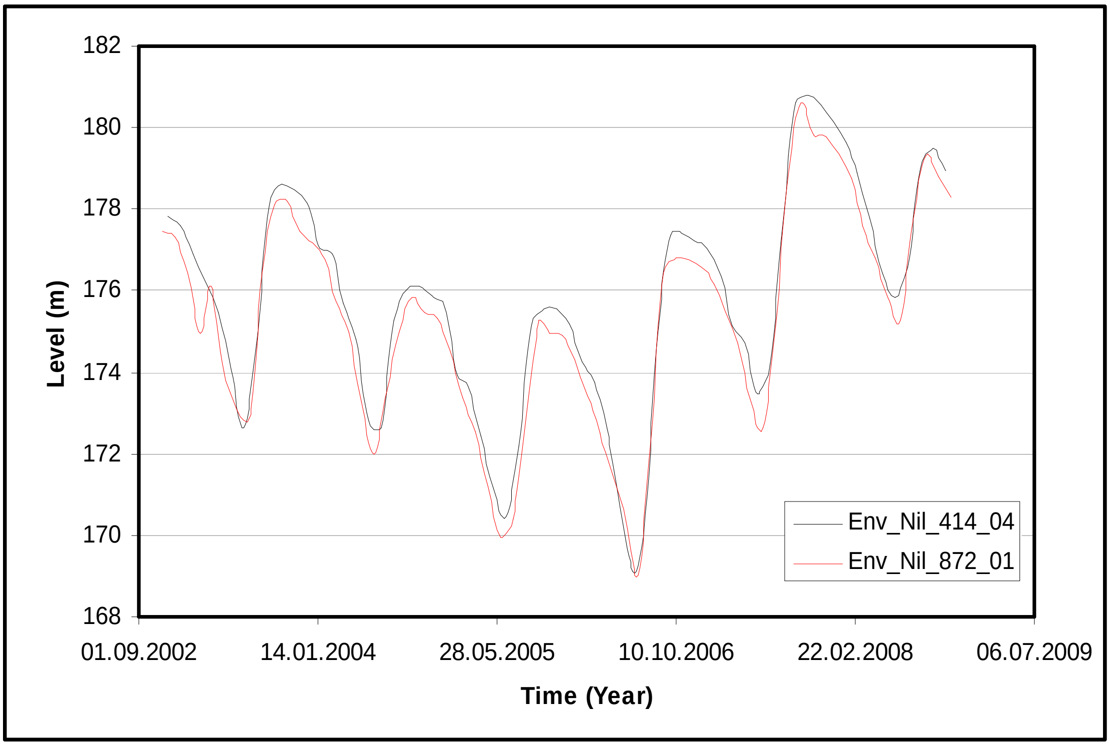


Figure 8.8. Comparison between the reservoir levels at tracks Env_Nil_414_04& Env_Nil_872_01.

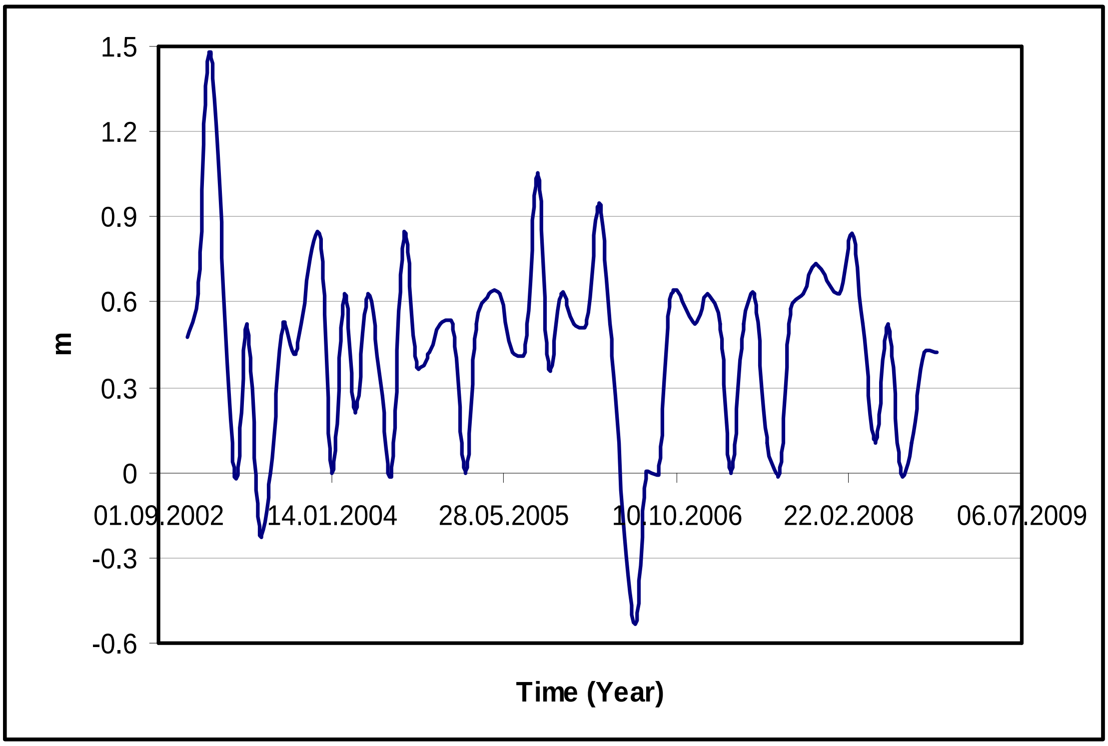


Figure 8.9. Reservoir levels variation between tracks Env_Nil_414_04& Env_Nil_872_01.

8.6 IMPACT OF LEVEL VARIATION ON THE RESERVOIR OPERATION

From the previous analysis of water level measurements by satellite altimetry to identify the level variation in the reservoir, it is found that there is a variation of about 0.43 m between the water level upstream Toshka spillway and at the AHD, so the discharges to the spillway should be modified based on the concluded level variation. Figure 8.10 shows water level upstream the AHD and corresponding values of the discharges to Toshka spillway in the two cases (before and after) taking impact of level variation in the consideration. In order to evaluate impact of level variation on the reservoir operation, the releases to Toshka spillway were modified according to figure 8.10 for both the policies (optimal operation rule (OPT) and current reservoir operation rule (CUR)).

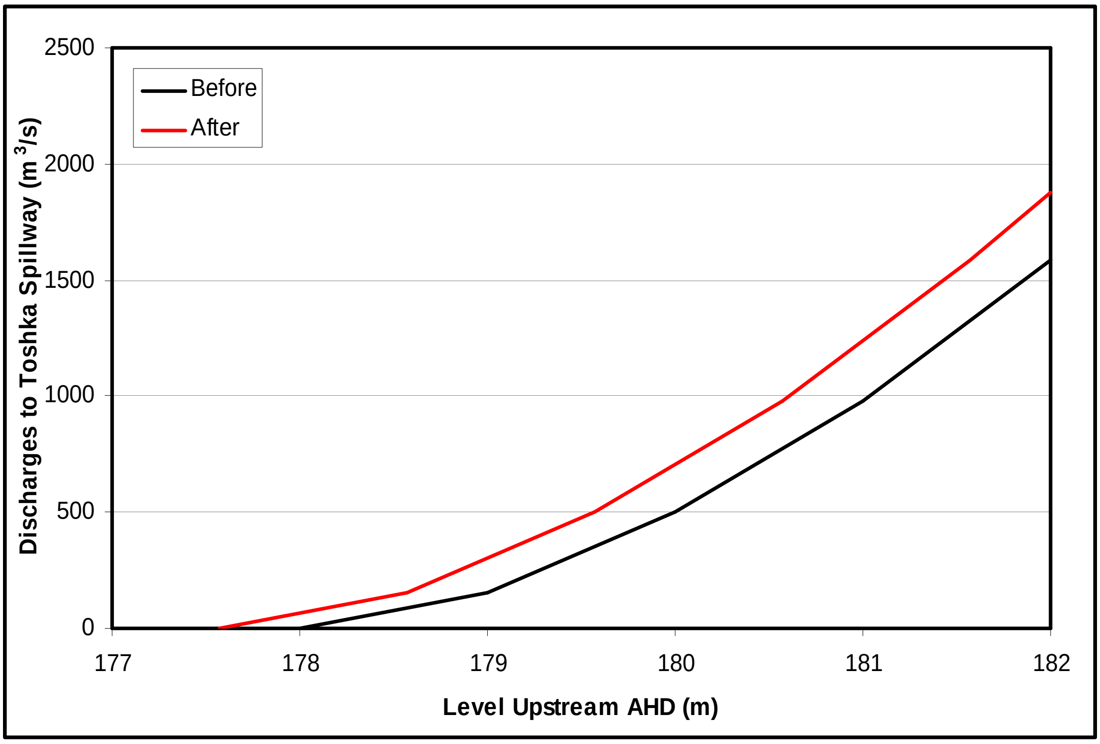


Figure 8.10. Water level upstream the AHD and discharges to Toshka spillway.

Period I in scenario IV was chosen to be the comparable period for two reasons, first, this period represents the maximum scenario of flood inflow, outflow discharge downstream the AHD and spillage to Toshka spillway, and the second reason, there is no spillage to Toshka spillway during periods II, III for the OPT policy. The reservoir performance under variation level impact was assessed relative to the following criteria:

- Discharges to Toshka spillway.
- Water supply releases.
- Evaporation losses.
- Hydropower production.

8.6.1 Impact of Level Variation on The discharges to Toshka Spillway

Table 8.1 summarizes the Statistical comparison of discharges to Toshka spillway during period I in scenario IV for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR). Several conclusions can be derived from the results presented in table 8.1., under the CUR policy and before taking the level variation in the estimations, the annual range of discharges to Toshka spillway varied between 0.03 – 12.18 BCM with a mean of about 3.41 BCM, standard deviation (STDV) of 2.92 BCM and occur in approximately 76 % of years (figure 8.11), after taking impact of level variation, the range of projected releases to the spillway grows to 0.09 – 14.54 BCM with a mean of about 4.68 BCM, standard deviation (STDV) of 3.60 BCM and occur in approximately 79 % of years.
On the other hand, impacts of the level variation on the discharges to Toshka spillway in the OPT policy are smaller than the CUR policy. For clarification, in case of taking the impact in the estimations, the annual mean and standard deviation (STDV) was 1.73 BCM and 2.01 BCM respectively, compared to a mean of about 1.39 BCM and standard deviation (STDV) of 1.62 BCM before taking impacts of level variation.

	CUR					OPT				
	Max.	Max.	Min.	Mean	STDV	Max.	Max.	Min.	Mean	STDV
	m^3/s	BCM/year				m^3/s	BCM/year			
Before	1419	12.18	0.03	3.41	2.92	549	4.09	0.06	1.39	1.62
After	1690	14.45	0.09	4.68	3.60	681	5.27	0.07	1.73	2.01

Table 8.1. Statistical comparison of discharges to Toshka spillway for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

Figure 8.12 presents the monthly discharges to Toshka spillway using the two different policies (OPT & CUR) for the two cases (before & after) taking impact of level variation. From this figure, it can be seen that the maximum releases to the spillway before taking the level variation impact for the CUR and the OPT policies was 1419 m^3/s (3.68 BCM/month) and 549 m^3/s (1.42 BCM/month) respectively, compared to 1690 m^3/s (4.38 BCM/month) and 681 m^3/s (1.76 BCM/month) after taking the level variation impact at the same period.

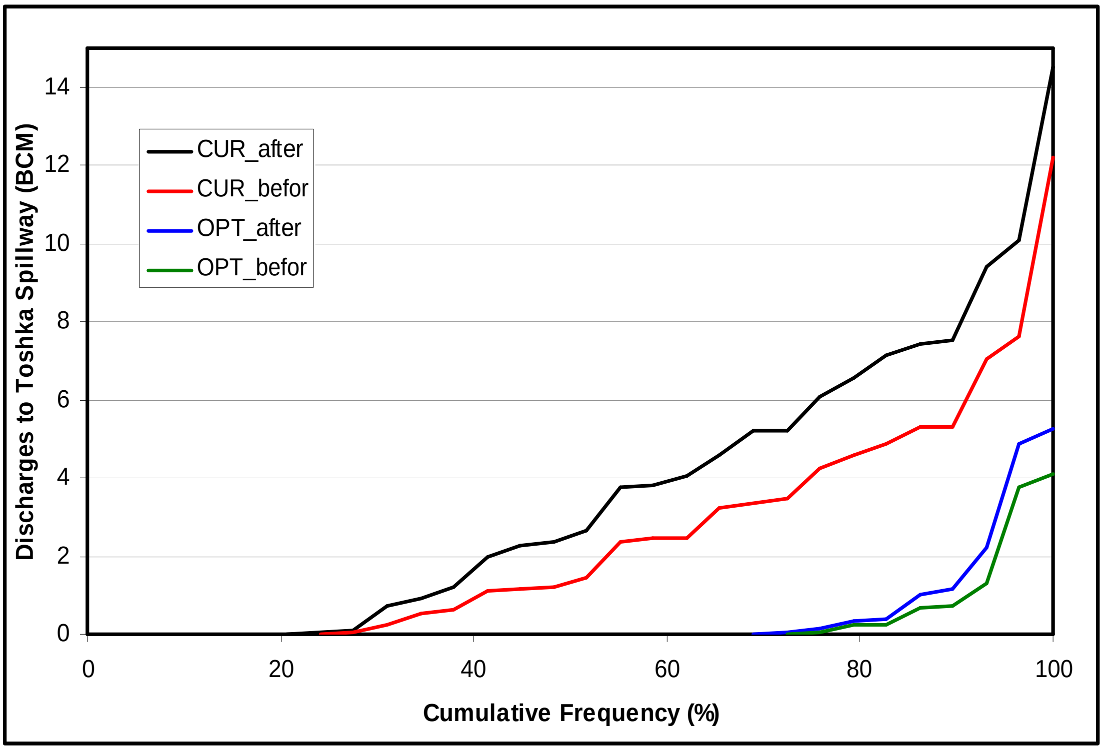


Figure 8.11. Comparison of frequency distribution curves of discharges to Toshka spillway for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

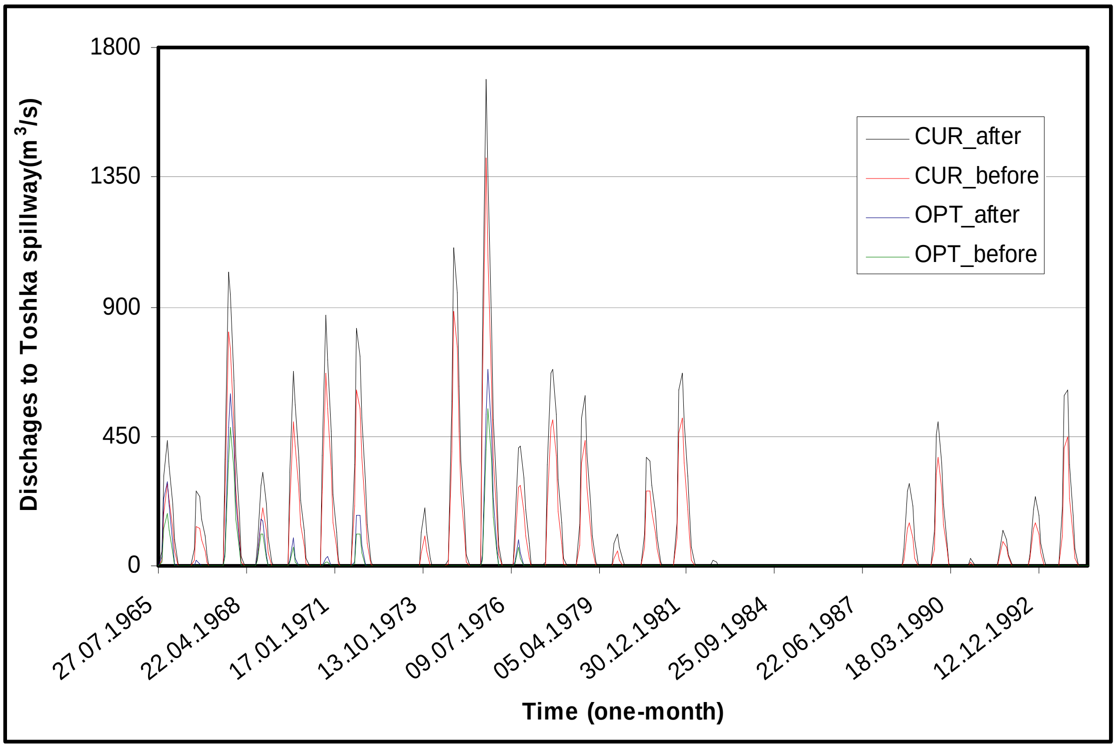


Figure 8.12. Comparison of discharges to Toshka spillway for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

8.6.2 Impact of Level Variation on The Water Supply Releases

Figure 8.13 shows the average annual withdrawal from the AHDR for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR). From this figure it can be concluded that according to the CUR policy, the mean annual withdrawal from the AHDR was 71.79 BCM before taking the level variation impact, this value decreases by 1.11 BCM to reach to 70.68 BCM after taking impact of level variation.
Impact of the level variation on the withdrawal from the reservoir for the OPT policy is approximately negligible, because the difference between the average annual withdrawal from the reservoir before and after taking the level variation impact is merely 0.11 BCM (figures 8.14 & 8.15).
Also, figure 8.15 presents comparison of the monthly releases downstream the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR). This figure shows how poorly impact of the level variation on the results for the OPT policy, but on the other side, for the CUR policy the maximum releases downstream the AHD reduced from 4592 m^3/s to reaches to 4186 m^3/s after taking the level variation impact.

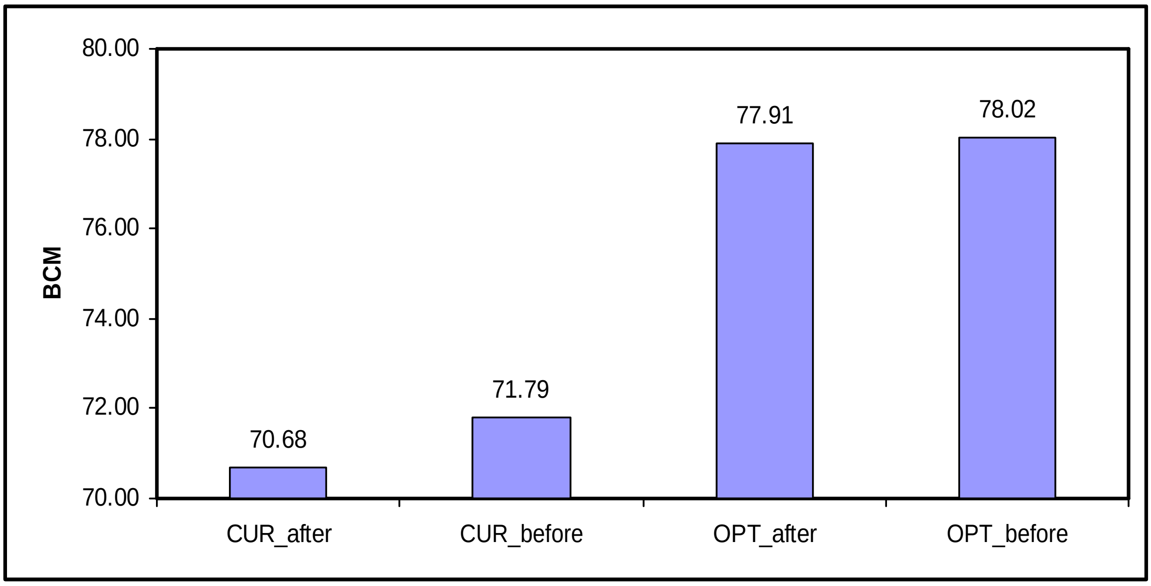


Figure 8.13. Comparison of average annual withdrawal from the AHDR for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

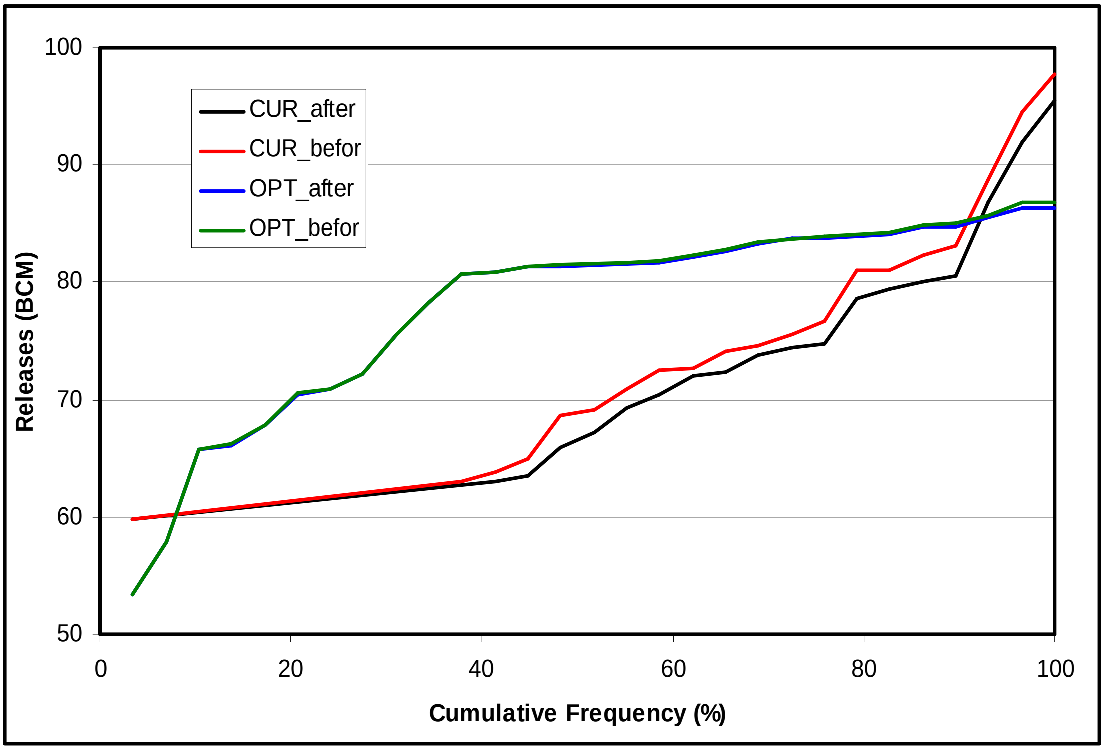


Figure 8.14. Comparison of frequency distribution curves of annual withdrawal from the AHDR for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

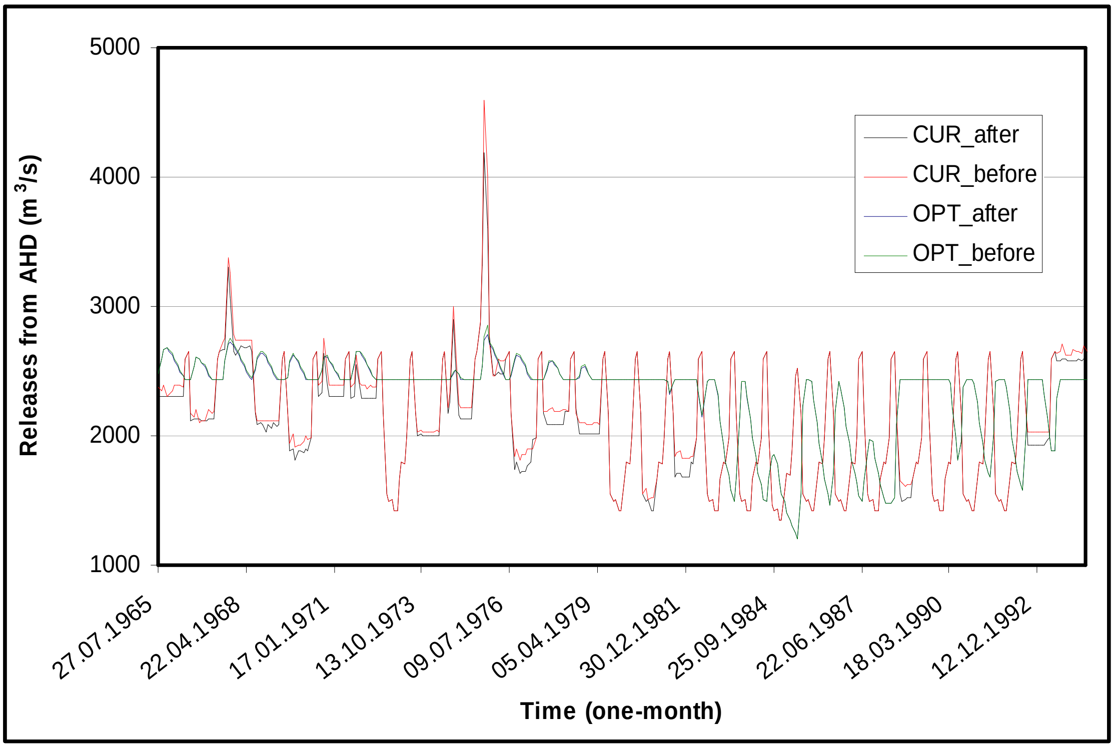


Figure 8.15. Comparison of the releases downstream the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

8.6.3 Impact of Level Variation on the evaporation Losses

Figure 8.16 presents the level upstream the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR). From these figure, it can be concluded that the water levels upstream the dam are not affected by the level variation impact approximately.
Therefore, it is expected that no change in the evaporation losses rates due to taking the effect of the level variation, and this is showed clearly in figure 8.17, which illustrates the of frequency distribution curves of evaporation losses from the AHDR for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

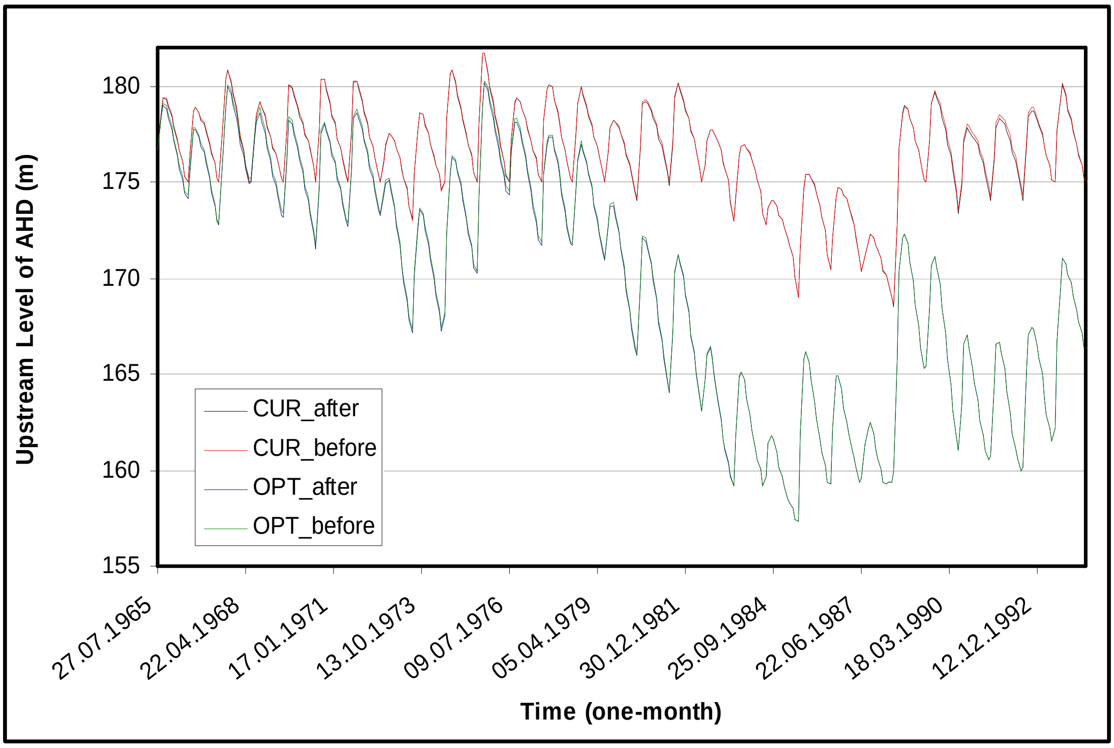


Figure 8.16. Comparison of the level upstream the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

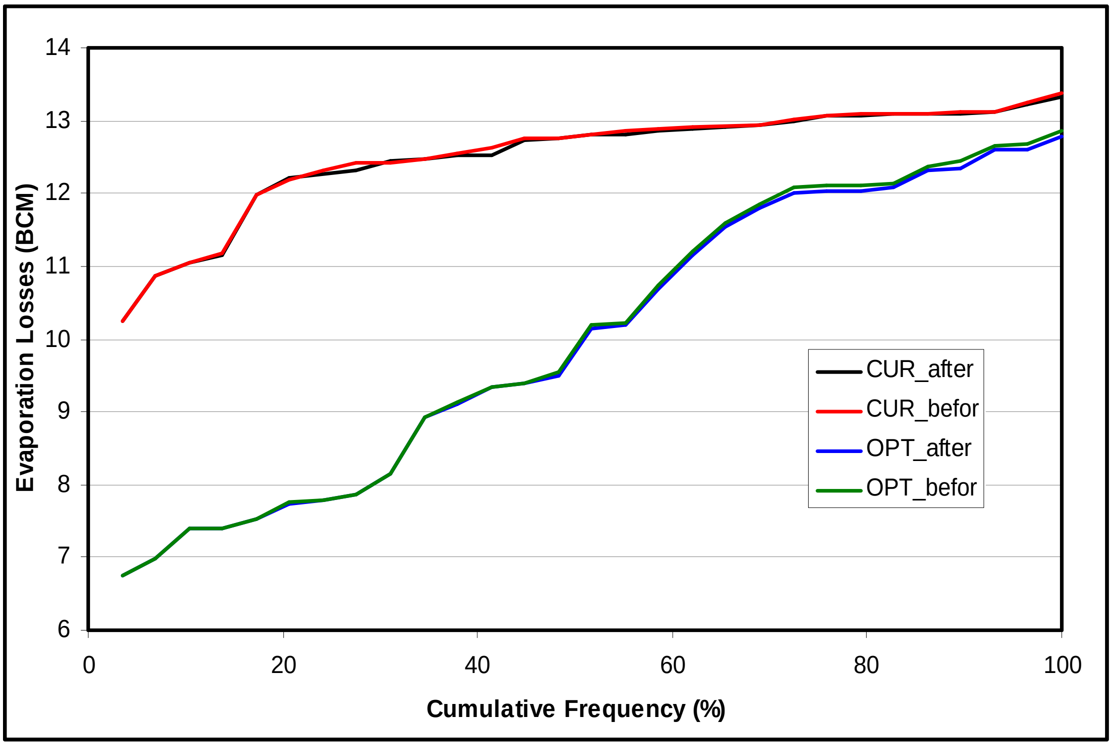


Figure 8.17. Comparison of frequency distribution curves of evaporation losses from the AHDR for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

8.6.4 Impact of Level Variation on The hydropower Production

The change in the hydropower production before and after taking the level variation impact is closely linked to the change in the releases downstream the AHD, and this is confirmed by Table 8.2., which displays Statistical comparison of hydropower production from the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR). Table 8.2 and figure 8.18 indicate that, for the CUR policy, annual hydropower production at the AHD before taking the level variation impact varied between 7920 – 13582 GWh with a mean of about 10181 GWh and standard deviation (STDV) of 1592 GWh. After taking impact of level variation, this values decrease to 7918 – 13204 GWh, 10009 GWh and 1483 GWh for the range and the mean and standard deviation (STDV), respectively.

	CUR				OPT			
	Max.	Min.	Mean	STDV	Max.	Min.	Mean	STDV
	GWh/year				GWh/year			
Before	13582	7920	10181	1592	12788	5558	10229	2075
After	13204	7918	10009	1483	12692	5557	10197	2050

Table 8.2. Statistical comparison of hydropower production from the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

As shown in figure 8.18, for the OPT policy, the change in the annual hydropower production at the AHD due to effect of the level variation is slight change, because the change in the reservoir levels and the releases downstream the AHD is approximately negligible.

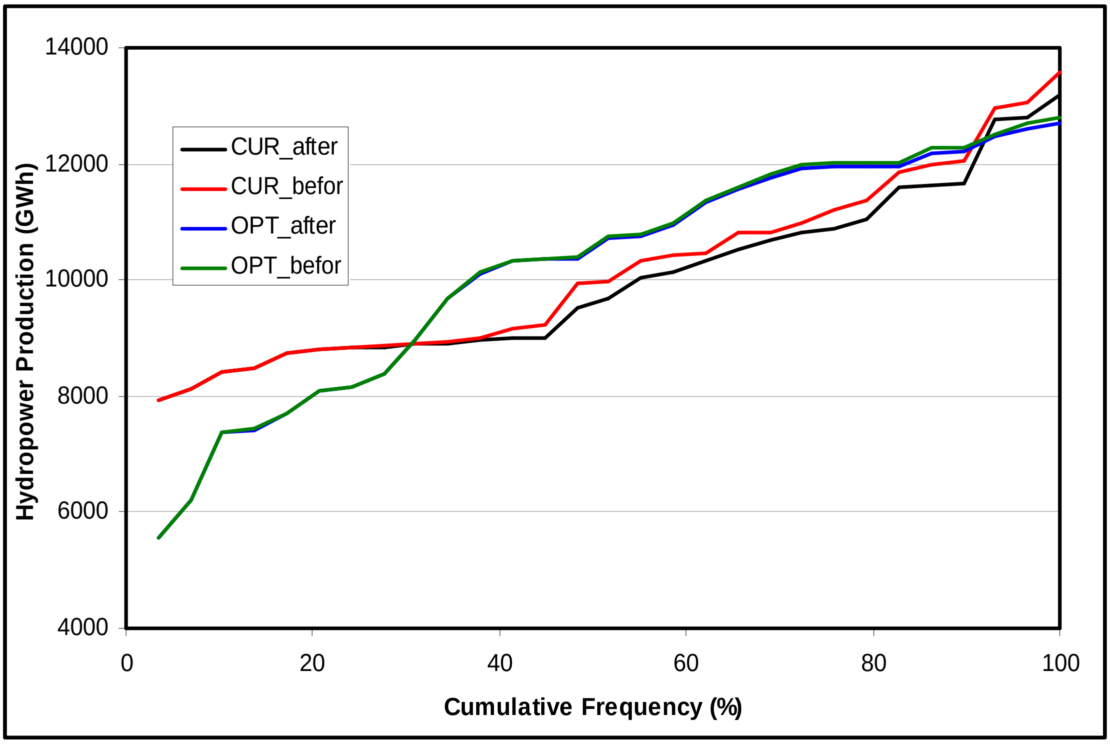


Figure 8.18. Comparison of frequency distribution curves of hydropower production from the AHD for the two cases (before & after) taking impact of level variation using the two different policies (OPT & CUR).

9 CONCLUSIONS & RECOMMENDATIONS

9.1 SCIENTIFIC CONTRIBUTION

The Aswan high dam reservoir (AHDR) case study is very important in its own right. It represents the long term storage reservoir for the water of the Nile flowing to Egypt. Therefore, special and national consideration must be given to the reservoir operation and development. A comprehensive new approach to analyses the potential future changes and related modifications of the infrastructure for the reservoir was developed. This culminated in several significant contributions:

- Development and calibration of a new simulation model for the AHDR using BlueM simulation modeling system for integrated river basin / reservoirs operation systems. The model has the ability to evaluate of the reservoir behavior under different hydrologic conditions and operation policies and any kind of structural changes;
- Demonstrated the relevance and importance of climate impact assessments to reservoir operation, including the implications for future water policy and management strategies;
- An optimization model to optimize the control strategies for the AHDR and determine the optimal releases policy was developed using the software BlueM;
- In order to improve the efficiency of the AHDR, a dynamic operating rule was devised which links the discharge from the reservoir to the current reservoir level. The objective of the dynamic operating rule is to reduce evaporation and spill losses from the reservoir, and thus increase Nile water availability. A comparison is made of existing operating policy for the AHD with that resulting from a dynamic operating policy;
- Monitoring the AHDR water level variations using satellite radar altimetry data to determine the impact of this variation on the reservoir operation for current and optimal operation rules.

9.2 SUMMARY OF ASSESSMENT FINDINGS

The assessments are carried out to demonstrate that climate scenarios can be used in conjunction with management models to assess the potential benefits of future development and management strategies that might mitigate adverse climate effects. It is hoped that the results of these assessments will lead to be more informed about the AHDR strategies for future development, adaptive management, and risk assessment.
Detailed assessment findings have been presented in Chapters 6, 7, and 8, this section serves to compile these findings:

- Egypt's average annual withdrawal from the AHDR is expected to increase due to climate change by 7.30 BCM early in the 21st century (2010-2039). However, Egypt might suffer significant shortfalls relative to historical average releases from the AHDR reaches to 3.30 BCM and 6.00 BCM by mid (2040-2069) and late (2070-2099) century, respectively;
- Hydropower production at the AHD is projected to increase early in the century to 117 percent of historical average production, but then decreasing to 91 and 80 percent of the historical mean for mid and late century, respectively;
- Under dry scenarios, Jonglei canal project increases the releases from the AHDR by 3.00 BCM, this value grows to 6.70 by the full implementation of Baro-Akobo project;
- Hydropower production at the AHD is expected to increase to 107 and 115 percent of current production by the full implementation of Jonglei canal project and Baro-Akobo project, respectively;
- There is a possibility to increase Nile water availability in order of 5 BCM/year approximately through a change in the AHDR operation from current operation policy to optimal operation policy;
- The results of the comparison between reservoir levels for some of the satellite tracks over the AHDR indicated a general decline in the water surface in the reservoir, specially between the water level at Toshk spillway and at the AHD. It is found that, there is average level variation between water level at Toshk spillway and at the AHD of about 0.43 m;
- Under wet scenario, the mean annual withdrawal from the AHDR decreased by 1.11 BCM after taking impact of level variation for current operation policy due to increasing spillage to Toshka spillway. However, the level variation impact on the withdrawal from the reservoir for optimal operation policy was approximately negligible. In addition, hydropower production at the AHD was affected by level variation effect for current operation policy, but the level variation impact on hydropower production at the AHD for optimal policy was also negligible.

9.3 CONCLUSION AND RECOMMENDATIONS

The assessments carried out in this work have resulted in a number of improvements that lead to more definitive conclusions:

- Inflow to the AHDR is very sensitive to any change in rainfall in the Nile basin;
- The study showed how climate changes affected the reservoir operation in case of flood or drought scenarios;
- Climate impact assessments on the AHDR operation have produced meaningful results that can now be incorporated in water management and policy-making considerations;
- For a dryer scenario, irrespective of the level of inflow reduction, Egypt might have to face water shortage;

- Importance of cooperation and coordination among Nile Basin countries to build water conservation projects that work to reduce the losses from the Nile and thus increase Nile water availability;
- A possibility of increase the releases from the AHDR through a change in reservoir operation;
- The proposed dynamic operating rule in this study was clearly superior in flood and drought mitigation;
- Impact of reservoir level variation should be taken into consideration to perform or develop simulation models for the AHDR;
- Reservoir adaptation to larger variability is very important, and may be one of the important steps to good water management;
- The results from this study provide an objective basis for decision makers to weight scenario outcomes;
- The scenarios and reservoir operation policies, which have been simulated in this study, have demonstrated the ability of BlueM modeling system to model complex reservoir systems and to be adaptable to any kind of structural changes. Due to the modular structure the program becomes flexible and for every kind of real component a model element can be configured and modified.

9.4 OUTLOOK FOR FUTURE WORK

This work can be expanded in various ways that involve the climate, hydrology, and water resources components as outlined below:

- Expand the reservoir operation assessments to include more demand, development, climate change and water management scenarios;
- Study the effect of hydraulic characteristics of the AHDR cross sections on the management of the reservoir;
- Besides using powerful modeling and optimization techniques, the efficiency of the derived reservoir operations also depends on the accuracy or uncertainty of input data. An important aspect that needs to be considered in the future work is how to handle uncertainties and stochasticity in the optimization process. Uncertainties and stochasticity are normally represented by multiple system states in an ensemble setting, which adds an additional computational challenge to the optimization problem;
- Study the impact of the integrated management of the constructed reservoirs on the Nile river, and its impact on reducing the losses from these reservoirs and thus the possibility of raising the efficiency of these reservoirs in the water supply and hydropower production;
- Toshka spillway plays an important role in the flood conditions, so it needs to be subject to further studies, which includes the impact of construction of some engineering structures (Barrages, weirs, flap gates,..…….etc.) along the spillway on the efficiency of the spillway;

- More water resources assessments would serve to highlight the socio-economic impacts of climate change for the Nile Basin, as well as the development and management strategies that would best mitigate adverse impacts.

REFERENCES

Abdel-Moteleb, M., and Saad, M. B. A., (2001), "Calibration Of An Ogee Weir," IAHR Congress, China.

Abu-Zeid, M. A., and El-Shibini, F. Z., (1997), "Egypt's High Aswan Dam," Water Resources Development, Vol. 13, No. 2, pp. 209- 217.

Agrawala, S., Moehner, A., El-Raey, M., Conway, D., Aalst, M. V., Hagenstad, M., and Smith, J., (2004), "Development And Climate Change In Egypt: Focus On Coastal Resources And The Nile," Envrionment Directorate, Environment Policy Committee, Working Party on Global and Structural Policies and Working Party on Development Co-operation and Environment, Organisation for Economic Co-operation and Development (OECD).

Ahmed, T. A., (December 1999), "Role Of The Aswan High Dam In Promoting The Nile River System Management," presented at the WCD Regional Consultation, Cairo, Egypt.

Ahmad, A. M., (2008), "Post-Jonglei Planning In Southern Sudan: Combining Environment With Development," Environment and Urbanization, Vol. 20, No. 2, 575-586, DOI 10.1177/0956247808096129.

Ahmed, A. M., Mohammed, A. A., and El-Otify A. M., (1989), "Field And Laboratory Studies On Nile Phytoplankton In Egypt, III Some Physical And Chemical Characteristics Of Aswan High Dam Lake (Lake Nasser)," Int. Revue ges. Hydrobiol., 74, 329-348.

Allam, M. N., and Allam, G. I., (2007), "Water Resources In Egypt: Future Challeges And Opportunities," International Water Resources Association, Volume 32, No. 2, P. 205-218.

Alnaggar, D., (December 2003), "Water Resources Management And Policies For Egypt," Workshop on Policies and Strategies Options for Water Management In Islamic Countries (Tehran).

Aly, A. I. M., Froehlich, K., Nada, A., Awad, M., Harnza, M., and Salem, M. W., (1993), "Study Of Environmental Isotope Distribution In The Aswan High Dam Lake (Egypt) For Estimation Of Evaporation Of Lake Water And Its Recharge To Adjacent Groundwater," Environmental Geochemistry and Health 15(1), page37.

Amer, M. H., Attia, M. B., Fahmy, H., and Tawfik, M., (December 2005), "Water Policy Issues Of Egypt," International Commission On Irrigation And Drainage (ICID), New Delhi.

An Assessment of the Intergovernmental Panel on Climate Change (IPCC), (November 2007), "Climate Change 2007: Synthesis Report," Valencia, Spain.

Antar, M. A., (December 2008), "Climate Change Experience," DWF –Copenhagen, Denmark.

Aquastat, (2005), "FAO's Information System On Water And Agriculture," *http://www.fao.org/ag/agl/aglw/aquastat/dbase/index.stm*.

Atiya, A. F., El-Shoura, S. M., Shaheen, S. I., and El-Sherif, M. S., (March 1999), “A Comparison Between Neural-Network Forecasting Techniques—Case Study: River Flow Forecasting,” IEEE Transactions on Neural Networks, Vol. 10, No. 2.

Attia, B. B., (February 2007), “Water Resources Management In Egypt “Challenges And Proposed Actions,” International Conference on "Water Management in the Islamic Countries", Tehran.

Awad, A. and Zohry, A., (20 - 22 December 2005), “The End Of Egypt Population Growth In The 21st Century: Challenges And Aspirations,” The 35th Annual Conference on Population and Development Issues: Current Situation & Aspirations, Cairo Demographic Center, Cairo, Egypt.

Ayana, E. K., (March 2007), “Validation Of Radar Altimetry Lake Level Data And It's Application In Water Resources Management,” Master Thesis, University of Twente, The Netherland.

Bach, M., Froehlich, F., Heusch, S., Huebner, C., Muschalla, D., Reu ner, F., and Ostrowski, M., (2009), “Bluem – A Free Software Package For Integrated River Basin Management,” Day of Hydrology, Kiel, Germany.

Barry, N., Gennero, M-C., Crétaux, J-F., Bergé-Nguyen, M., Calmant, S., and Cazenave, A., (2-3 May 2000), “Time Water Levels On Rivers And Lakes Using Jason-2 Altimetry Data,” DORIS Days Meeting Toulouse, France.

Baeck, Hammel, and Schwefel, (1997), “Evolutionary Computation: Comments On The History And Current State,” IEEE Transactions on Evolutionary Computation 1, no. 1: 3-17.

Beyene, T., Lettenmaier, D. P., and Kabat, P., (2009), “Hydrologic Impacts Of Climate Change On The Nile River Basin: Implications Of The 2007 IPCC Climate Scenarios,” Climatic Change, DOI 10.1007/s10584-009-9693-0.

Buchanan, R. B., and Bras, R. L., (July 1982), “A Study Of A Real Time Adaptive Closed-Loop Reservoir Control Algorithm,” Optimal Allocation of Water Resources (Proceedings of the Exeter Symposium, IAHS Publ. no. 135).

Burn, D. H., and Simonovic, S. P., (1996), “Sensitivity Of Reservoir Operation Performance To Climatic Change,” Water Resources Management 10, 463-478.

Chang, F., Chen, L., and Chang, L., (2005), “Optimizing The Reservoir Operating Rule Curves By Genetic Algorithms,” Hydrological Processes 19, pp. 2277- 2289.

Chaves, P., and Kojiri, T., (2003), “Multi-Objective Storage Reservoir Operation Under Uncertainty,” Annuals of Disas. Prev. Res. Inst., Kyoto Univ., No.46 B, Japan.

Cheng, K., Yeh, H., and Liou, C., (June 2000), “Comparative Study Of Drought Prediction Techniques For Reservoir Operation,” American Water Resources Association, Vol. 36, No. 3.

Conway, D., (2005), "From Headwater Tributaries To International River: Observing And Adapting To Climate Variability And Change In The Nile Basin," Global Environmental Change 15, 99–114.

Conway, D., and Hulme, M., (1996), "The Impacts Of Climate Variability And Future Climate Change In The Nile Basin On Water Resources In Egypt," International Journal of Water Resources Development, 13(3): 277-296.

Conway, D., and Hulme, M., (1993), "Recent Fluctuations In Precipitation And Runoff Over The Nile Sub-Basins And Their Impact On Main Nile Discharge," Climatic Change 25, 127-151.

Deb, (2001), "Multi-Objective Optimization Using Evolutionary Algorithms," Chichester: John Wiley & Sons.

Dumont, H. J., (2009), "The Nile: Origin, Environments, Limnology And Human Use," Springer Netherlands, Volume 89, ISBN 978-1-4020-9725-6 (Print) 978-1-4020-9726-3 (Online).

Eastern Nile Subsidiary Action Program (ENSAP), (last updated Monday, 11 June 2007), "Baro-Akobo Multi- Purpose Water Resources," *http://ensap.nilebasin.org/index.php?option=com_content&task=view&id=39&Itemid=131*

Elarabawy, M., Attia, B., and Tosswell, P., (1998), "Water Resources In Egypt: Strategies For The Next Century," Journal of Water Resources Planning and Management, PP. 310-319.

El Bastawesy, M., Arafat, S., and khalaf, F., (2007), "Estimation Of Water Loss From Toshka Lakes Using Remote Sensing And GIS," 10th AGILE International Conference on Geographic Information Science, Aalborg University, Denmark.

EL-Sersawy, H., (Mach 2007), "Development Of Decision Support System (DSS) For High Aswan Dam Reservoir Sedimentation," International Congress on River Basin Management, Turkey.

El-Kady, M., and El-Shibini, F., (2004), "Integrated Water Resources Management; The System of Systems for Sustainable Development: The Egyptian Experience," International Conf. on Water Resources & Arid Environment.

Elsawwaf, M., Willems, P., Pagano, A., and Berlamont, J., (June 2009), "Evaporation Estimates From Nasser Lake, Egypt, Based On Three Floating Station Data And Bowen Ratio Energy Budget," Theor Appl Climatol, DOI 10.1007/s00704-009-0168-z.

EL-Sersawy, H., (2005), "Sediment Deposition Mapping In Aswan High Dam Reservoir Using Geographic Information System (GIS)," Ninth International Water Technology Conference, IWTC9, Sharm El-Sheikh, Egypt.

El Shaer, H. M., Rosenzweig, C., Iglesias, A., Eid, H. M., and Hiller, D., (1997), “Impact Of Climate Change On Possible Scenarios For Egyptian Agriculture In The Future,” Mitigation and Adaptation Strategies for Global Change, Vol. 1, Number 3, pp. 233-250.

El-Shafie, A., Abdin, A. E., Noureldin, A., and Taha, M. R., (2008), “Enhancing Inflow Forecasting Model At Aswan High Dam Utilizing Radial Basis Neural Network And Upstream Monitoring Stations Measurements,” Water Resour Manage.

Elshamy, M. E., (2008), “Assessing The Hydrological Performance Of The Nile Forecast System In Long Term Simulations,” Nile Basin Water Engineering Scientific Magazine, Vol.1.

ESA "European Space Agency", (2009), http://tethys.eaprs.cse.dmu.ac.uk/RiverLake/region/15N030E?loc=?95,259.

Fahmy, H., (March 2001), “Modification And Re-Calibration Of The Simulation Model Of Lake Nasser,” Water International, Volume 26, Issue 1, pages 129 - 135.

Farmani, Walters, and Savic, (2006). “Evolutionary Multi-Objective Optimization Of The Design And Operation Of Water Distribution Network: Total Cost Vs. Reliability Vs. Water Quality,” Journal of Hydroinformatics 8, no. 3: 165-179.

Froehlich, F., and Ostrowski, M., (2009), “Dynamic Reservoir Operation For Reducing Dam-Induced Hydrologic Alteration,” The 7th International Symposium on Ecohydraulics, Chile.

Froehlich, F., Robert, D., Dirk, M., Ostrowski, M., and Pohl, R., (28.04.-02.05.2008), “Re-Operation Of Multi-Purpose Reservoirs For Economic And Environmental Benefits,” 28th USSD Annual Meeting and Conference, Portland, Oregon, USA.

Gennero, M-C., Barry, N., Crétaux, J-F., and Cazenave, A., (2-3 May 2000), “An Hydrological Data Base Of Time Water Levels On Rivers And Lakes From Altimetry,” DORIS Days Meeting Toulouse, France.

Georgakakos, A. P., (September 8 – 10, 2004), “Decision Support Systems For Integrated Water Resources Management With An Application To The Nile Basin,” Fifth IFAC Workshop on Time-Delay Systems, Katholieke Universiteit Leuven, Belgium.

Georgakakos, A., (July 2002), “Climate Projections, Uncertainty, And Scenarios For Impact Assessments: Water Resources Planning And Mgt.,” Georgia Water Resources Institute / Georgia Tech.

Georgakakos, A., and Yao, H., (March 2001), “Value Of Climate Forecasts For The Management Of Lake Lanier,” Georgia Water Resources Institute.

Georgakakos, A. P., and Yao, H., (1999), “Nile River Basin Impacts To Climate And Anthropogenic,” Georgia Water Resources Institute (GWRI), Atlanta.

Georgakakos, A. P., Yao, H., and Miller, F., (May 1997), “A Decision Support System For The High Aswan Dam,” Waterpower.

Ghaas (1998), (visited on July 2008), “The Global River Discharge Database (Rivdis V1.1),” Internet site: *http://www.rivdis.sr.unh.edu/cgi-bin/ViewSite?SITE=01487*.

Hall, W. A., and Dracup, J. A., (1970), “Water Resources Systems Engineering,” McGraw-Hill series in water resources and environmental engineering.

Hamza, W., and Mason, S., (2004), “Water Availability And Food Security Challenges In Egypt,” International Forum on Food Security Under Water Scarcity in the Middle East: Problems and Solutions”, Como, Italy.

Hassan, R., Hekal, N., and Mansor, N., (2007), “Evaporation Reduction From Lake Naser Using New Environmentally Safe Techniques,” Eleventh International Water Technology Conference, IWTC11 Sharm El-Sheikh, Egypt.

Hassan, W. E. M. A., (2008), “Study Of Sedimentation Inside The Lake Of The High Dam,” Master Thesis, Alazher University, Cairo, Egypt.

Hawker, P., (Mach 2000), “A Review Of The Role Of Dams And Flood Management,” World Commission on Dams.

Ibrahim, A. M., (1984), “The Nile: Description, Hydrology, Control And Utilisation,” Dr W. Junk Publishers, The Hague. Printed in the Netherlands, Hydrobiologia 110, 1-13.

International Commission on Irrigation and Drainage (ICID), (2005), “Water Policy Issues Of Egypt,” New Delhi.

Ismail, S. S., and Aziz, M., (2005), “Toshka Spillway Barrages Stability Analysis,” Ninth International Water Technology Conference, IWTC9, Sharm El-Sheikh, Egypt.

Karamouz, M., and Araghinejad, S., (2008), “Drought Mitigation Through Long-Term Operation Of Reservoirs: Case Study,” Irrigation and Drainage Engineering, Vol. 134, No. 4.

Karyabwite, D. R., (2000), “Water Sharing In The Nile River Valley,” UNEP/DEWA/GRID - Geneva.

Kashef, A., (1981), “Technical And Ecological Impacts Of The High Aswan Dam,” Journal of Hydrology, 53, 73-84.

Khalil, M. A., Santos, F. A. M., Moustafa, S. M., and Saad, U. M., (2009), “Mapping Water Seepage From Lake Nasser, Egypt, Using The VLF-EM Method: A Case Study,” Geophysics and Engineering 6, 101–110.

Kim, U., Kaluarachchi, J. J., and Smakhtin, V. U., (2008), “Climate Change Impacts On Hydrology And Water Resources Of The Upper Blue Nile River Basin, Ethiopia,” Research Report 126, International Water Management Institute (IWMI), Colombo, Sri Lanka.

LEGOS "Laboratorie d'Etudes en Geophysique et Oceanographie Spatiales" (France), (2009), http://www.legos.obs-mip.fr/soa/hydrologie/hydroweb/.

Loucks, D. P., Beek, E. V., Stedinger, J. R., and Dijkman, J. P. M., (2005), “Water Resources Systems Planning And Management,” ISBN 92-3-103998-9, UNESCO and WL | Delft Hydraulics.

Loz n, J. L., Grassl, H., Hupfer, P., Menzel, L., and Schoenwiese, C., (2007), “Global Change: Enough Water For All?,” Wissenschaftliche Auswertungen/GEO, ISBN 3-9809668-2-8, Hamburg, Germany.

Mahmoud, M. K., (April 2005), “Improvement The Efficiency Of Toshka Spillway Canal,” Water Science, Issue No. 37.

Mason, S. A., (2003), “From Conflict To Cooperation In The Nile Basin,” A dissertation for the degree of Doctor of Sciences, Swiss Federal Institute of Technology Zurich, Switzerland.

McCuen, R. H., (1998), “Hydrologic Analysis And Design,” Prentice-Hall, ISBN 0- 13- 134958-9, New Gersy, USA.

Metwaly, M., Khalil, M., Al-Sayed, E., and Osman, S., (2006), “A Hydrogeophysical Study To Estimate Water Seepage From Northwestern Lake Nasser, Egypt,” Geophysics And Engineering 3, 21–27.

Mimikou, M. A., Hadjisavva, P. S., Kouvopoulos, Y. S., and Afrateos, H., (1991), “Regional Climate Change Impacts: II. Impacts On Water Management Works,” Hydrological Sciences 36,3, 6.

Ministry of Electricity and Energy "MEE", (2005), “Annual Report 2004/2005,” Cairo, Egypt.

Ministry of Water Resources and Irrigation (MWRI), (2005), “Water For The Future, National Water Resources Plan 2017 (NWRP),” Cairo, Egypt.

Mohamed, Y. A., Van den Hurk, B. J. J. M., Savenije, H. H. G., and Bastiaanssen, W. G. M., (2005), “Hydroclimatology Of The Nile: Results From A Regional Climate Model,” Hydrol. Earth Syst. Sci. Discuss., 2, 319–364.

Moussa, A., Soliman, M., and Aziz, M., (2001), “Environmental Evaluation For High Aswan Dam Since Its Construction Until Present,” Sixth International Water Technology Conference, IWTC, Alexandria, Egypt.

Nicol, A., (2003), “The Nile: Moving Beyond Cooperation,” UNESCO/IHP/WWAP.

Nile Basin Capacity Building Network ‘NBCBN’, (2005), “Floods And Their Influence On The Nile River System.”

Nile Basin Capacity Building Network ‘NBCBN’, (2005), “Flood And Catchment Management.”

Nile Basin Capacity Building Network ‘NBCBN’, (2005), “Assessment Of The Current State Of The Nile Basin Reservoir Sedimentation Problems.”

Okoth-Owiro, A., (2004), “The Nile Treaty, State Succession And International Treaty Commitments: A Case Study Of The Nile Water Treaties,” Nairobi.

Organisation for Economic Co-operation and Development (OECD), (2004), “Development And Climate Change In Egypt: Focus On Coastal Resources And The Nile.”

Osman, H. M., Osman, M. K., and Karmy, A. S., (2003), “Silting-Up Of High Aswan Dam: Design, Investigation And Removal Of Deposits,” Dam Maintenance and Rehabilitation, Lianos et al. (eds), Swets & Zeitlinger, Lisse, ISBN 90 5809 534 7.

Osman, H. M., (December 1999), “Hydro Development In Egypt, Lessons From High Aswan Dam,” presented at the WCD Regional Consultation, Cairo, Egypt.

Ostrowski, M., (October 2009), “Ingenieurhydrologie II,” A textbook, Darmstadt University of technology, Darmstadt, Germany.

Ostrowski, M., (1992), “Ein universeller Baustein zur Simulation hydrologischer Prozesse,” Wasser und Boden.

Ostrowski, M., (2010), “A generic Mathamatical Formulation of A piecewise Linear Approximation of Nonlinear Hydrological Storage Processes,” (in preparation).

Pahl-Wostl, C., (2007), “Transitions Towards Adaptive Management Of Water Facing Climate And Global Change,” Water Resour Manage 21, 49–62, DOI 10.1007/s11269-006-9040-4.

Pahl-Wostl, C., Sendzimir, J., Jeffrey, P., Aerts, J., Berkamp, G., and Cross. K., (2007), “Managing Change Toward Adaptive Water Management Through Social Learning,” Ecology and Society (E&S), 12(2), 30.

Ragab, R., and Prudhomme, C., (2002), “Climate Change And Water Resources Management In Arid And Semi-Arid Regions: Prospective And Challenges For The 21st Century,” Biosystems Engineering, 81 (1), 3-34.

Raghunath, H. M., (2006), “Hydrology,” New Age International (P) Limited, Publishers, ISBN (10): 81-224-2332-9, 4835/24, Ansari Road, Daryaganj, New Delhi.

Rani, D., Moreira, M. M., and Mourato, S., (2009), “Preliminary Analysis Of Alvito-Odivelas Reservoir System Operation Under Climate Change Scenarios,” Option Me'diterrane'ennes, series A. No. 80.

Rashad, S. M., and Ismail, M. A., (2000), “Environmental-Impact Assessment Of Hydro-Power In Egypt,” Applied Energy 65 (285-302).

Rayan, A., Khalil, H., Fernandes, R. M. S., Mahmoud, S., and Tealeb, A., (September 2008), “Evaluation Of The Crustal Deformations In The Northern Region Of Lake Nasser (Egypt) Derived From 8 Years Of GPS Campaign Observations,” WEGENER, Darmstadt, Germany.

Reddy, M. J., and Kumar, D. N., (2006), “Optimal Reservoir Operation Using Multi-Objective Evolutionary Algorithm,” Water Resources Management 20, pp. 861- 878.

Rohde, F. G., and Naparaxawong, K., (May 1981), “Modified Standard Operation Rules For Reservoirs,” Journal of Hydrology, vol. 51, issue 1-4, pp. 169-177.

Saad, M. B., (2002), “Nile River Morphology Changes Due To The Construction Of High Aswan Dam In Egypt,” Ministry of Water Resources and Irrigation, Egypt.

Saad, M. B., (2008), “Nile River Flood Control: Egyptian Experiences,” International Center for Water Hazard and Risk Management (ICHARM), Japan.

Sadek, M. F., Shahin, M. M., and Stigter, C. J., (1997), “Evaporation From The Reservoir Of The High Aswan Dam, Egypt: A New Comparison Of Relevant Methods With Limited Data,” Theor. Appl. Climatol. 56, 57-66.

Sadek, N., and Aziz, M., (2005), “Flood Management Of Lake Nasser After The New Toshka Barrages Construction,” Ninth International Water Technology Conference, IWTC9, Sharm El-Sheikh, Egypt.

Sadek, N., (2006), “River Nile Flood Forecasting And Its Effect On National Projects Implementation,” Tenth International Water Technology Conference, IWTC10, Sharm El-Sheikh, Egypt.

Sayed, M. M., and Kamal, M. M., (2006), “Flood Evaluation And Management After The High Dam Reservoir,” Dams and Reservoirs, Societies and Enviroment in 21st Century – Berga et al (eds) Taylor & Francis Group, Londen, ISBN 0415404231.

Sechi, G. M., and Sulis, A., (2009), “Operative Indicators For Drought Mitigation Tools In Multireservoir Systems,” Option Me'diterrane'ennes, series A. No. 80.

Shahin, M. M. A., “An Overview Of Reservoir Sedimentation In Some African River Basins,” Sediment Problems: Strategies for Monitoring, Prediction and Control (Proceedings of the Yokohama Symposium, July 1993), LAHS Publ. no. 217, 1993.

Shalash, S., (1982), “Effects Of Sedimentation On The Storage Capacity Of The High Aswan Dam Reservoir,” Hydrobiologia 92, 623-639.

Shalash, S., (June 1980), “The Effect Of The High Aswan Dam On The Hydrological Regime Of The River Nile,” IAHS-AISH Publ. no. 130, Proceedings of the Helsinki Symposium.

Sharaf El Din, S. H., (March 1977), “Effeet Of The Aswan High Dam On The Nile Flood And On The Estuarine And Coastal Circulation Pattern Along The Mediterranean Egyptian Coast,” Limnology and Oceanography, Vol. 22, No. 2, pp. 194-207.

Shenouda, W. K., Hassouna, M. E., and Hawson, H., (1984), “Egypt's Aswan High Dam: Dinner Address Presented On December 17, 1956: Discussion,” Canadian Geotechnical Journal, 21, pp. 735-748.

Simonovic, S. P., Fahmy, H., and El-Shorbagy, A., (1997), “The Use Of Object-Oriented Modeling For Water Resources Planning In Egypt,” Water Resources Management 11: 243–261.

Smith, S. E., Mancy, K. H., Latif, A. F. A., and Fosnight, E. A., (August 1983), “Assessment And Monitoring Of Sedimentation In The Aswan High Dam Reservoir Using Landsat Imagery,” Hydrological Applications of Remote Sensing And Remote Data Transmission (Proceedings of The Hamburg Symposium). IAHS Publ. No. 145.

Soltani, M. A., Karimi, A., Bazargan-Lari, M. R., and Shirangi, E., (2008), “Stochastic Multi-Purpose Reservoir Operation Planning By Scenario Optimization And Differential Evolutionary Algorithm,” Applied Sciences 8 (22), 4186-4191.

Springuel, I., and Belal, A., (2001), “A Case Study On Ecotourism In The Wadi Allaqi Biosphere Reserve,” UNESCO, Division of Ecological Science, Egypt.

Strzepek, K. M., Yates, D. N., and El Quosy, D., (1996), “Vulnerability Assessment Of Water Resources In Egypt To Climatic Change In The Nile Basin,” Climate Research, Vol. 6: 89-95.

Strzepek, K. M., and Yates, D. N., (2000), “Responses And Thresholds Of The Egyptian Economy To Climate Change Impacts On The Water Resources Of The Nile River,” Climatic Change 46: 339–356.

Strzepek, K. M., Yates, D. N., Yohe, G., Tol, R., and Mader, N., (2001), “Constructing “Not Implausible” Climate And Economic Scenarios For Egypt,” Kluwer Academic Publishers. Printed in the Netherlands, Integrated Assessment 2: 139–157.

Strzepek, K. M., Yohe, G., Tol, R., and Rosegrant, M., (June 2006), “The Value Of The High Aswan Dam To The Egyptian Economy,” Working Paper FNU-111, JEL Classification C68, O13, Q25.

Strzepek, K. M., and McCluskey, A., (July 2007), “The Impacts Of Climate Change On Regional Water Resources And Agriculture In Africa,” The World Bank, Development Research Group, Sustainable Rural and Urban Development Team.

Strzepek, K., and Mccluskey, A., (July 2006), “District Level Hydroclimatic Time Series And Scenario Analyses To Assess The Impacts Of Climate Change On Regional Water Resources And Agriculture In Africa,” Special Series On Climate Change And Agriculture In Africa ISBN 1-920160-01-09.

Sutcliffe, J. V., and Parks, Y. P., (1999), “The Hydrology Of The Nile,” The International Association of Hydrological Sciences, IAHS Special Publication no. 5, ISBN 1-901502-75-9, UK.

Tatano, H., Kawai, H., and Okada, N., (1992), “Optimal Operation Model Of A Single Reservoir With Drought Duration Explicitly Concerned,” Stochastic Hydrol. Hydraul. 6, 123-134.

Tate, E., Sutcliffe, J., Conway, D., and Farquharson, F., (August 2004), “Water Balance Of Lake Victoria: Update To 2000 And Climate Change Modelling To 2100,” Hydrological Sciences–Journal–des Sciences Hydrologiques, 49(4).

Teodora, C., Wueest, A., and Wehrli, B., (March 2006), “Independent Review Of The Environmental Impact Assessment For The Merowe Dam Project (Nile River, Sudan),” Eawag "Aquatic Research", Switzerland.

Thompson, K. O., (January 1981), “Agricultural Vs. Hydropower Tradeoffs In The Operation Of The High Aswan Dam,” Master Thesis, University of Santa Clara, USA.

Tidwell, A. C., (December 2006), “Assessing The Impacts Of Climate Change On River Basin Management: A New Method With Application To The Nile River,” A Dissertation in Partial Fulfillment of the Requirements for the Degree Doctor of Philosophy in the School of Civil and Environmental Engineering, Georgia Institute of Technology.

U.S. Army Corps of Engineers, (October 1997), “Engineering And Design Hydrologic Engineering Requirements For Reservoirs,” Manual No. 1110-2-1420, Washington.

Volker, A., and Henry, J. C., (1988), “Side Effects Of Water Resources Management,” IAHS Publication No. 172, ISBN 0-947571-46-9.

Whittaker, W., Confesor, R., Griffith, S. M., Faere, R., Grosskopf, S., Steiner, J. J., Mueller-Warrant, G. W., and Banowetz, G. M., (2009), “A Hybrid Genetic Algorithm For Multiobjective Problems With Activity Analysis-Based Local Search,” European Journal of Operational Research 193, pp. 195- 203.

Whittington, D., and Guariso, G., (1983), “Water Management Models In Practice: A Case Study Of Aswan High Dam,” Elsevier Sientiefic Publishing Company, ISBN 0-444-42156-4.

Whittington, D., Wu, X., and Sadoff, C., (2005), “Water Resources Management In The Nile Basin: The Economic Value Of Cooporation,” Water Policy 7, 227-252.

WL | Delft Hydraulics, (2004), “Technical Assistance Services For Lake Nasser Flood And Drought Control/Integration Of Climate Change Uncertainty And Flooding Risk,” project description.

World Commission on Dams (WCD), (November 2000), “Dams And Development; A New Framework For Decision-Making,” London.

Wu, X., and Whittington, D., (2006), “Incentive Compatibility And Conflict Resolution In International River Basins: A Case Study Of The Nile Basin,” Water Resources Research, Vol. 42, W02417, Doi:10.1029/2005wr004238.

Yao, H., and Georgakakos, A. P., (June 2003), “Nile Decision Support Tool River Simulation And Management,” Georgia Water Resources Institute (GWRI), Atlanta.

Yates, D. N., and Strzepek, K. M., (April 1998), “Modeling The Nile Basin Under Climatic Change,” Journal of Hydrologic Engineering, Vol. 3, No. 2.

Yousef, S., and Osman, H., (September 2000), “The Great Nile Floods Of 1998 And 1999 ; Sucessful Forecasts Using Solar Terrestrial Relations And Real Data,” ICEHM2000, page 250- 262, Cairo University, Egypt.

APPENDIX A: SCENARIO ASSESSMENTS

A.1 DEVELOPMENT SCENARIO I

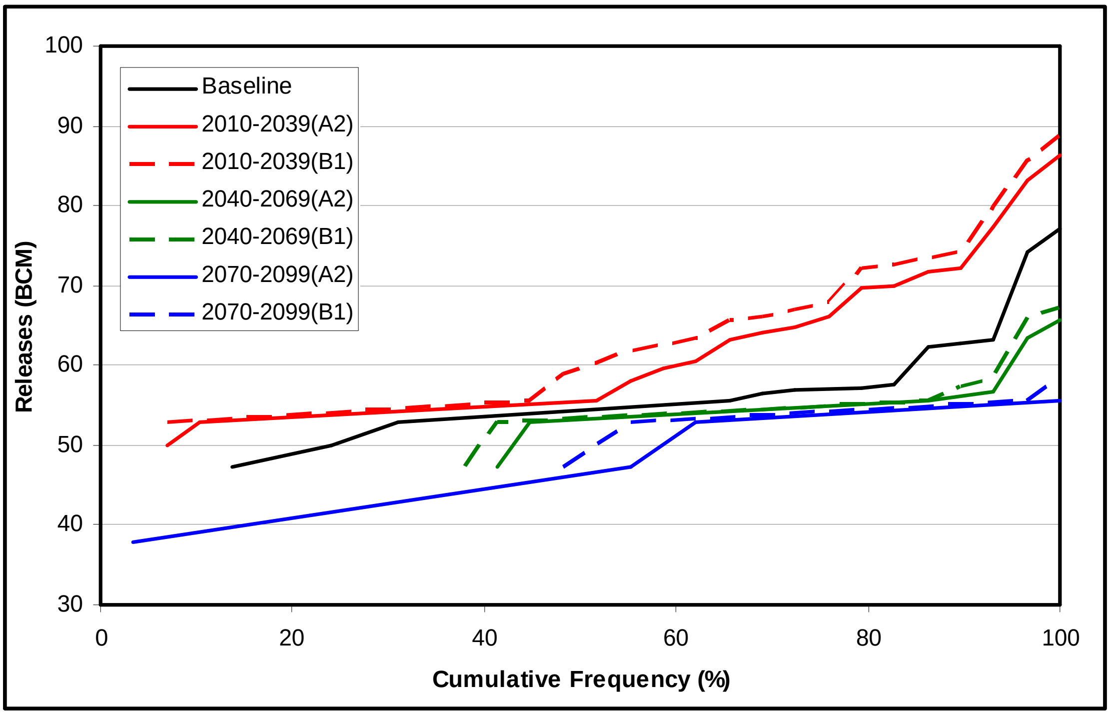


Figure A.1(a). Frequency curve of annual withdrawal from the AHDR for scenario I.

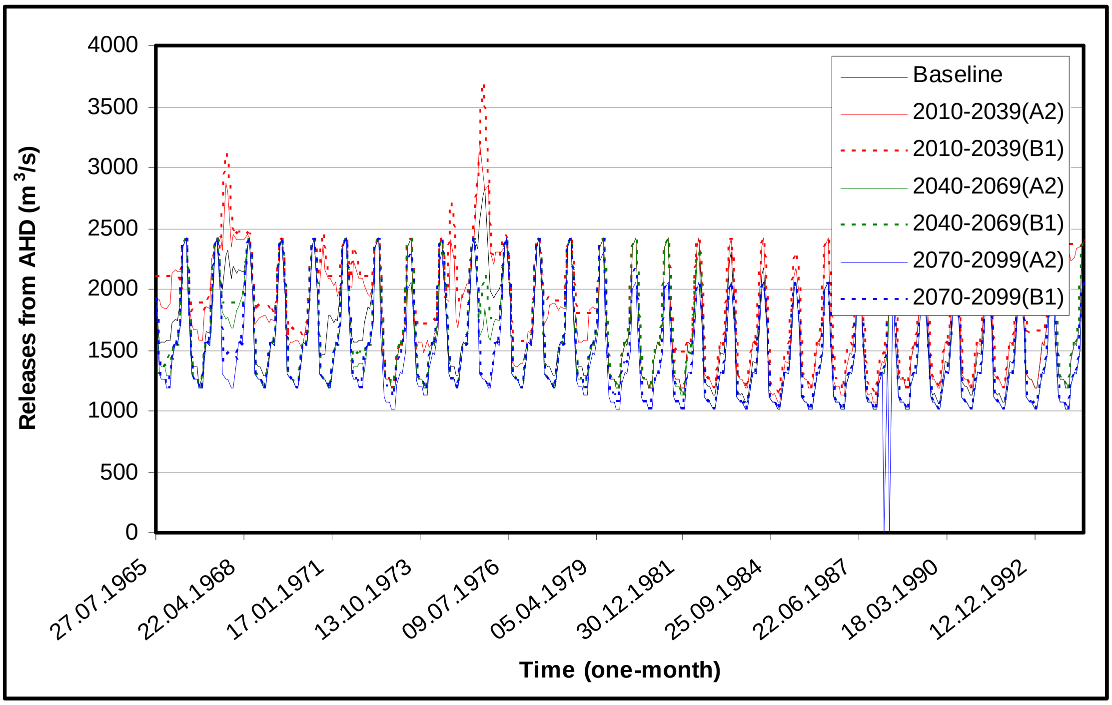


Figure A.1(b). Monthly releases from the AHD for scenario I.

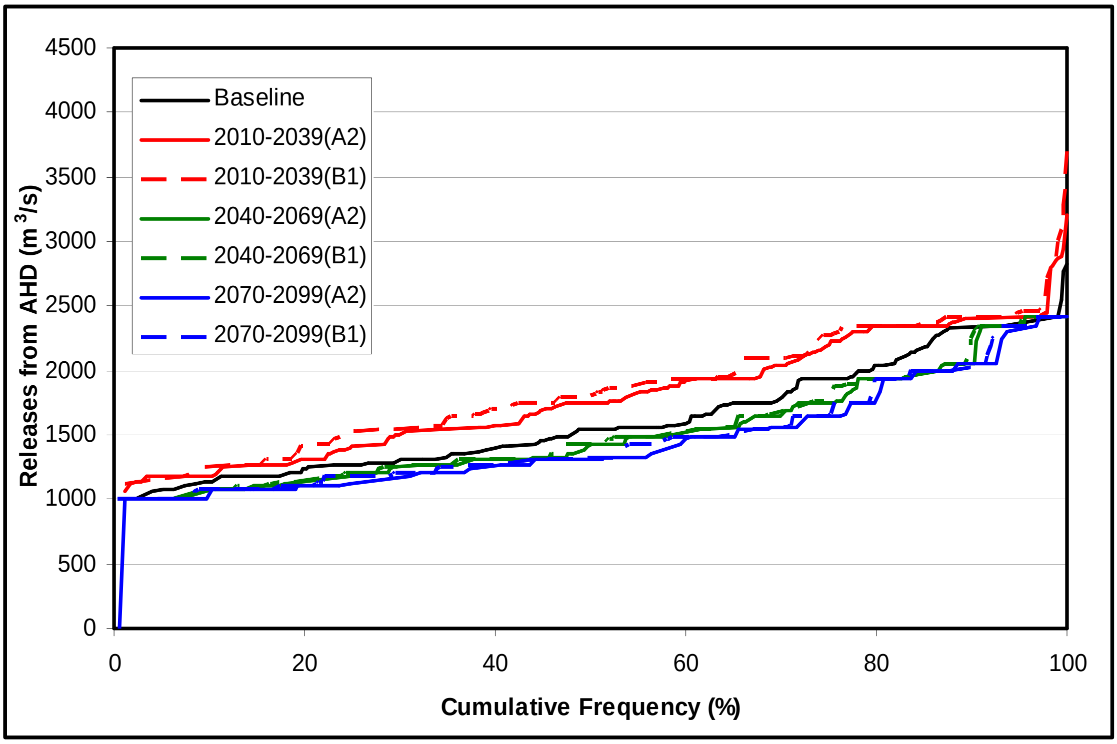


Figure A.1(c). Frequency curve of releases from the AHD for scenario I.

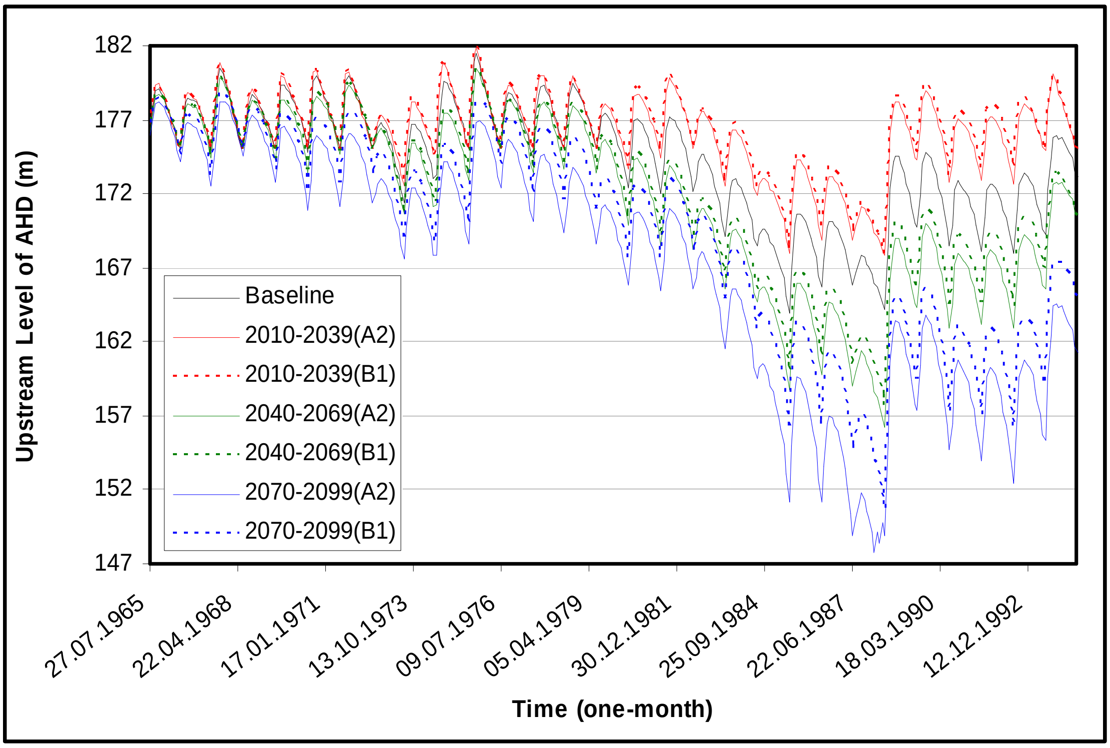


Figure A.1(d). Upstream levels of the AHD for scenario I.

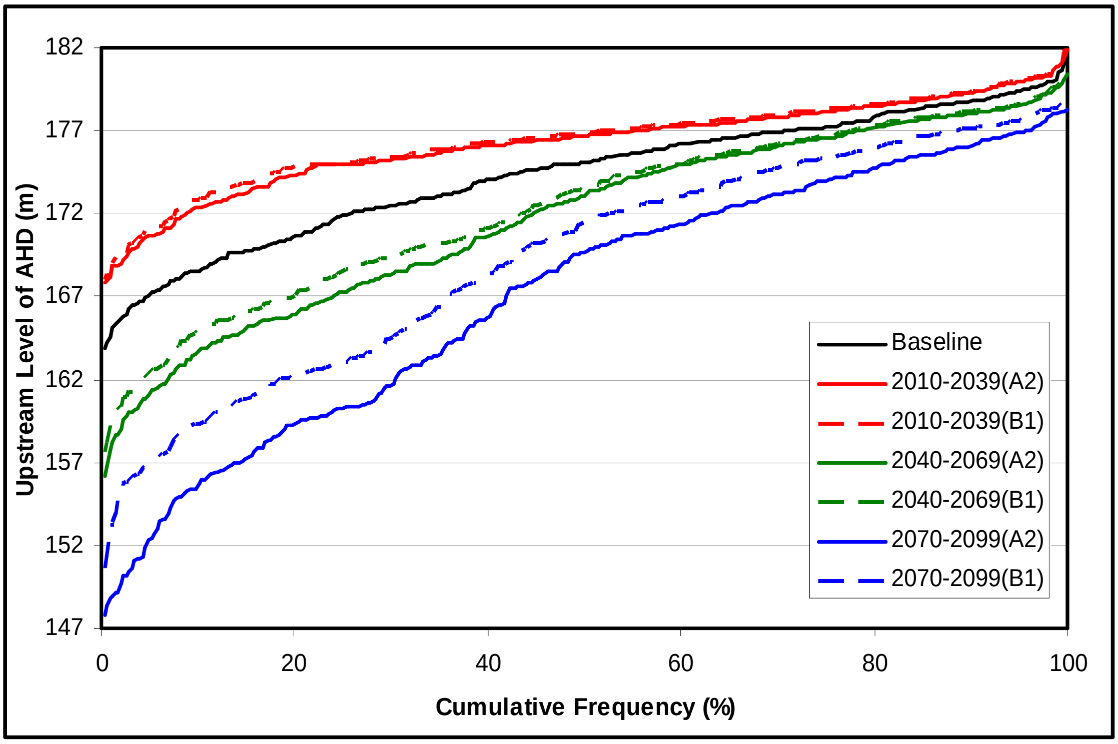


Figure A.1(e). Frequency curve of the AHD upstream levels for scenario I.

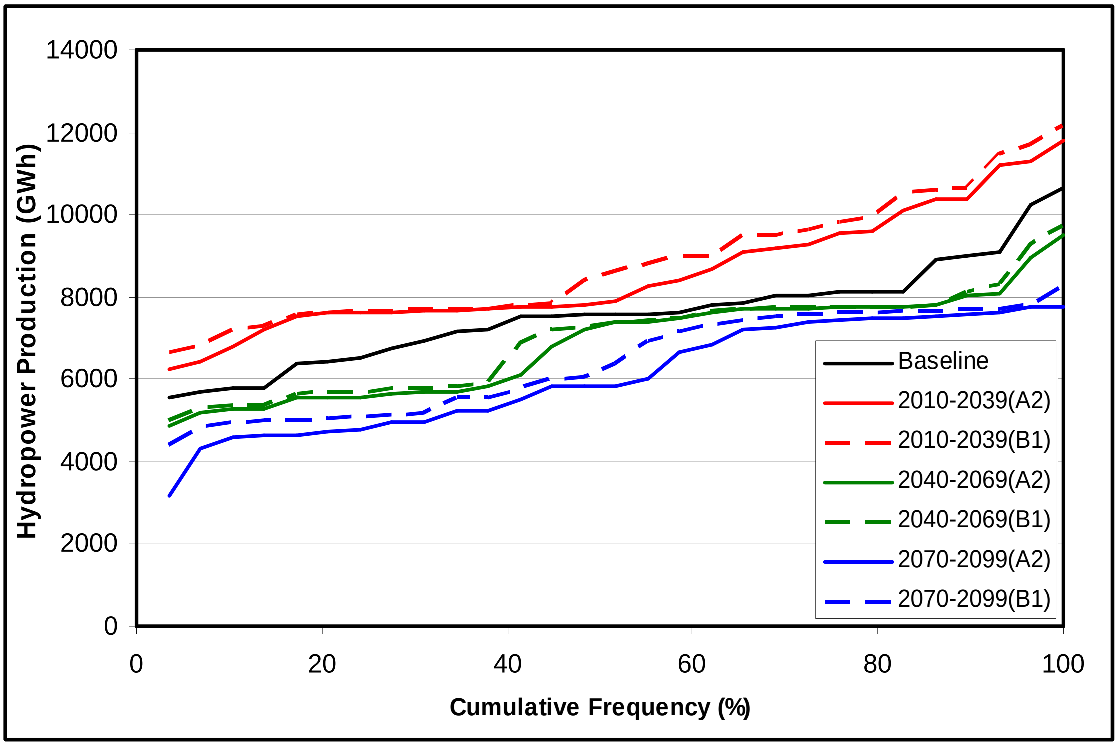


Figure A.1(f). Annual hydropower production frequency curve for scenario I.

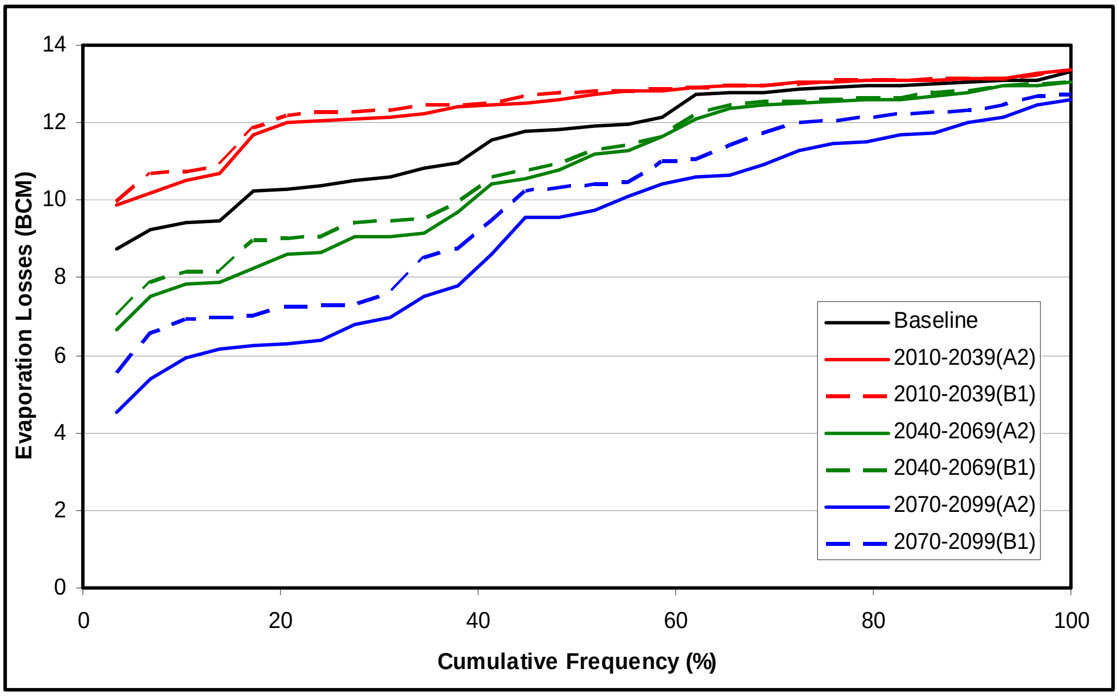


Figure A.1(g). Annual evaporation losses frequency curve for scenario I.

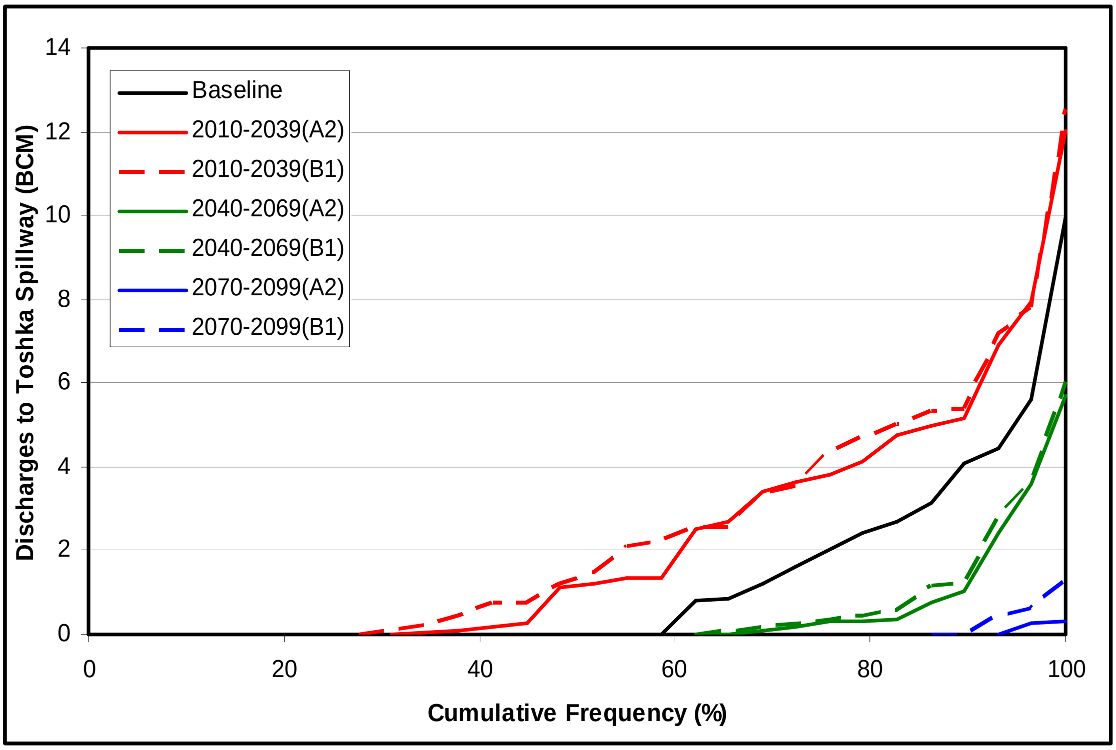


Figure A.1(h). Annual Toshka spillway discharges frequency curve for scenario I.

A.2 DEVOLOPMENT SCENARIO II

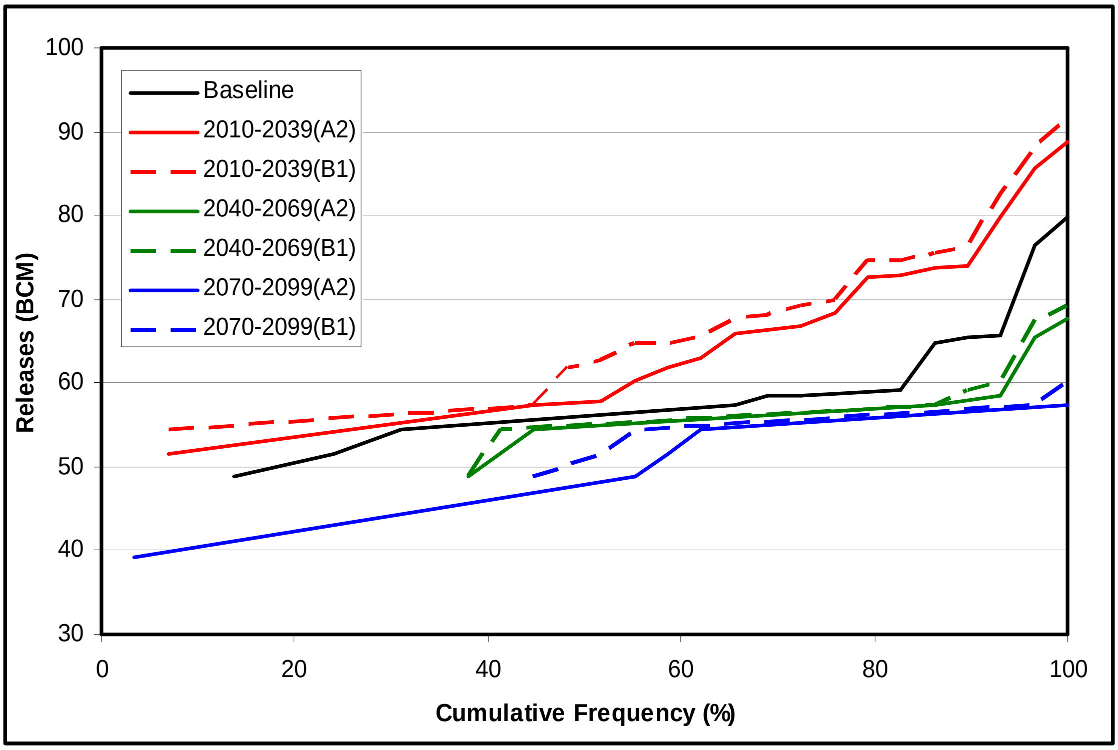


Figure A.2(a). Frequency curve of annual withdrawal from the AHDR for scenario II.

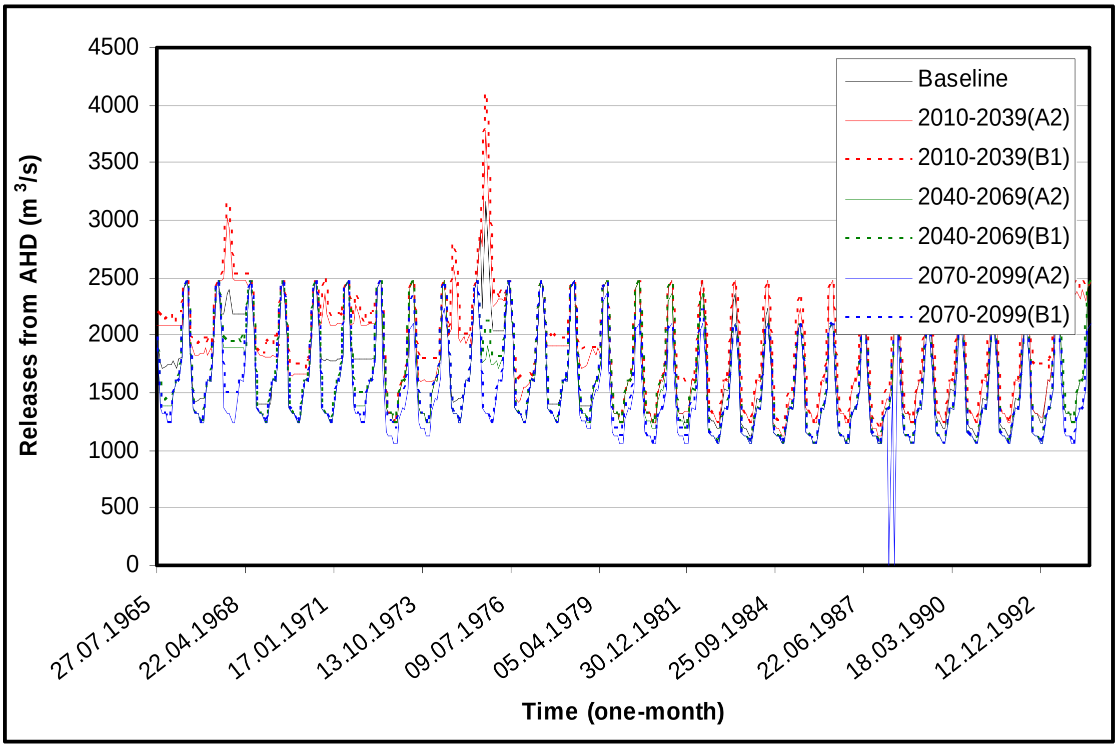


Figure A.2(b). Monthly releases from the AHD for scenario II.

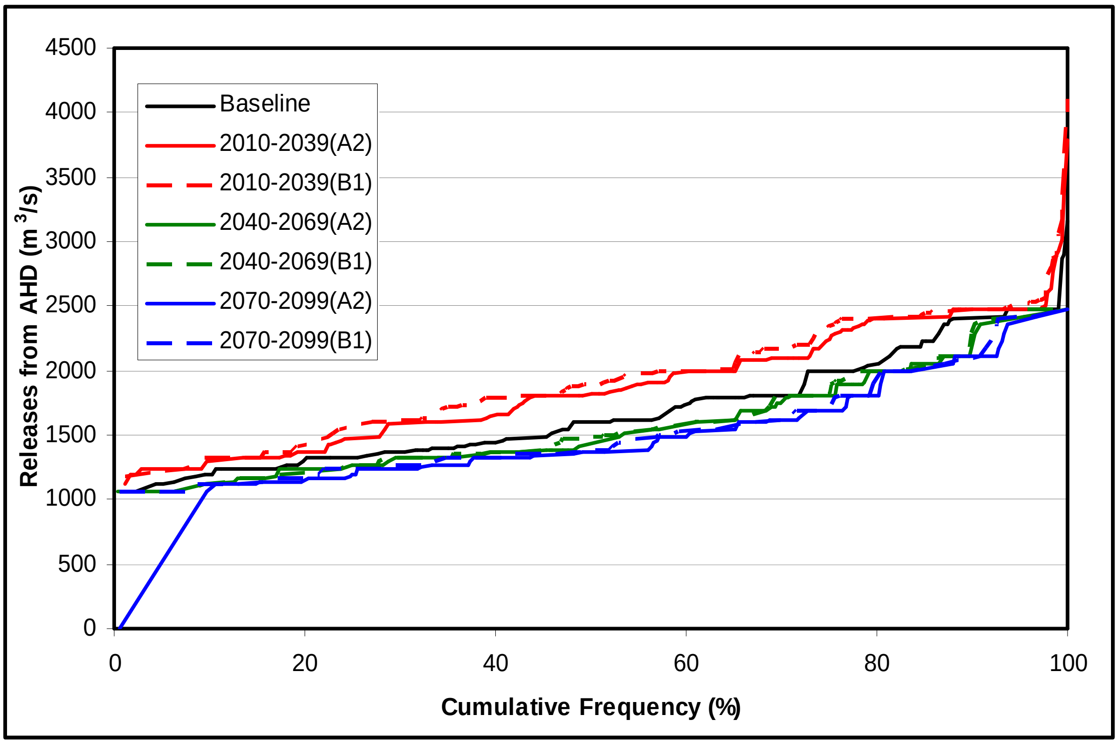


Figure A.2(c). Frequency curve of releases from the AHD for scenario II.

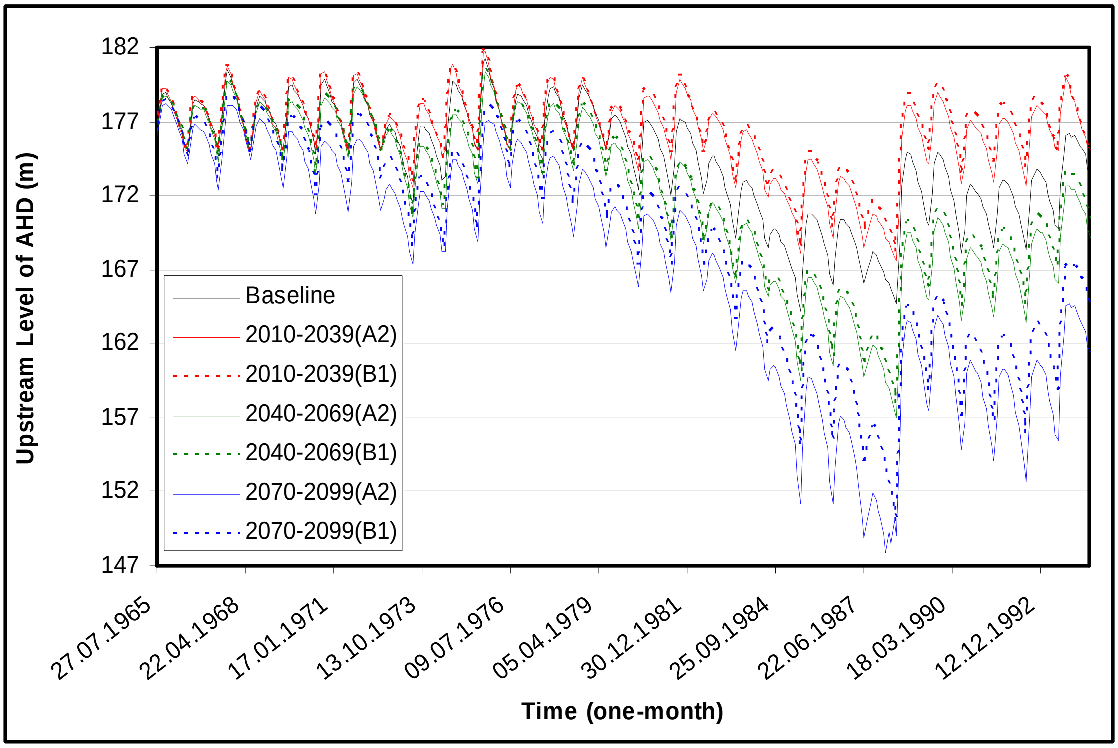


Figure A.2(d). Upstream levels of the AHD for scenario II.

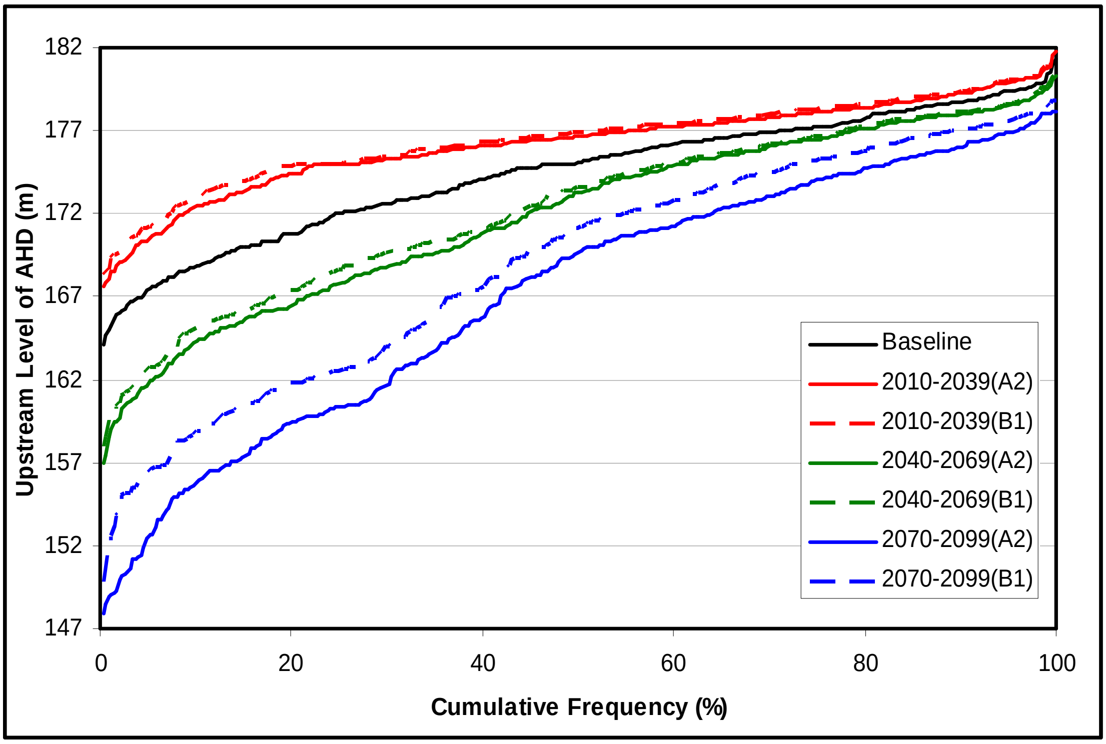


Figure A.2(e). Frequency curve of the AHD upstream levels for scenario II.

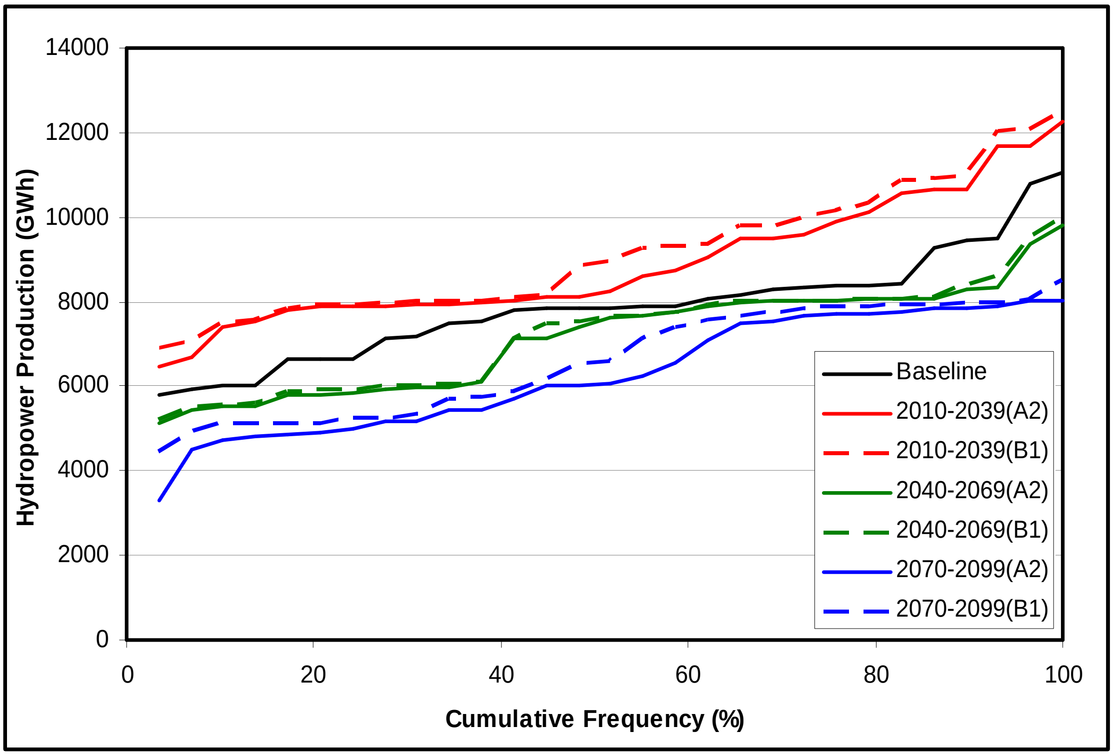


Figure A.2(f). Annual hydropower production frequency curve for scenario II.

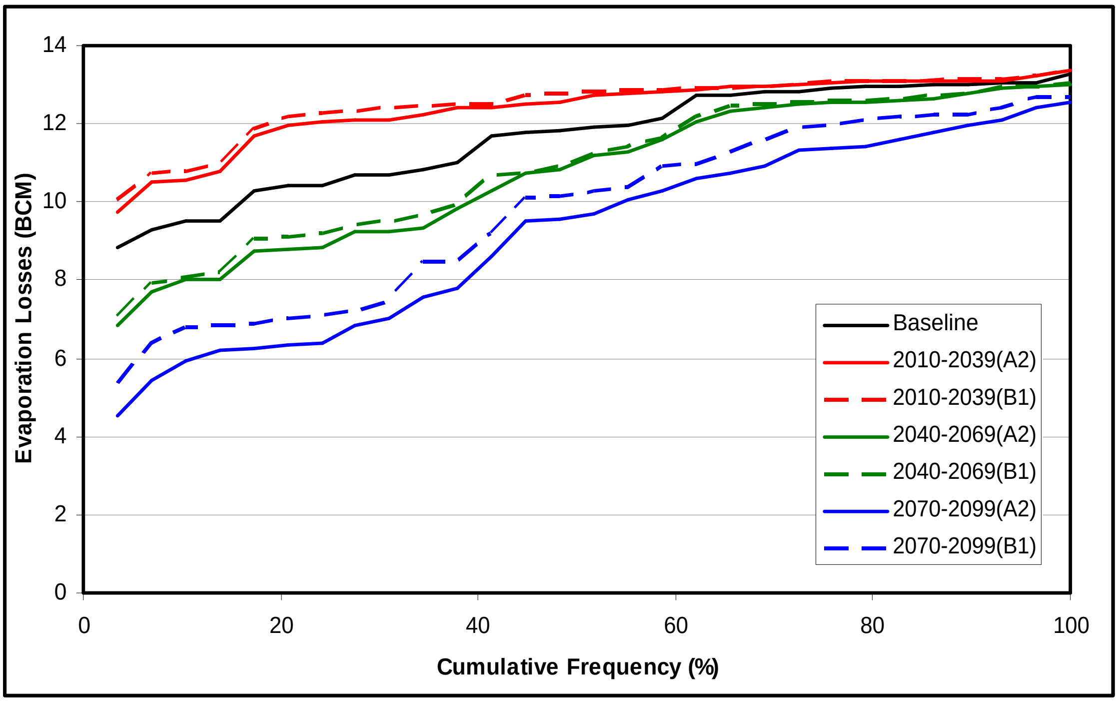


Figure A.2(g). Annual evaporation losses frequency curve for scenario II.

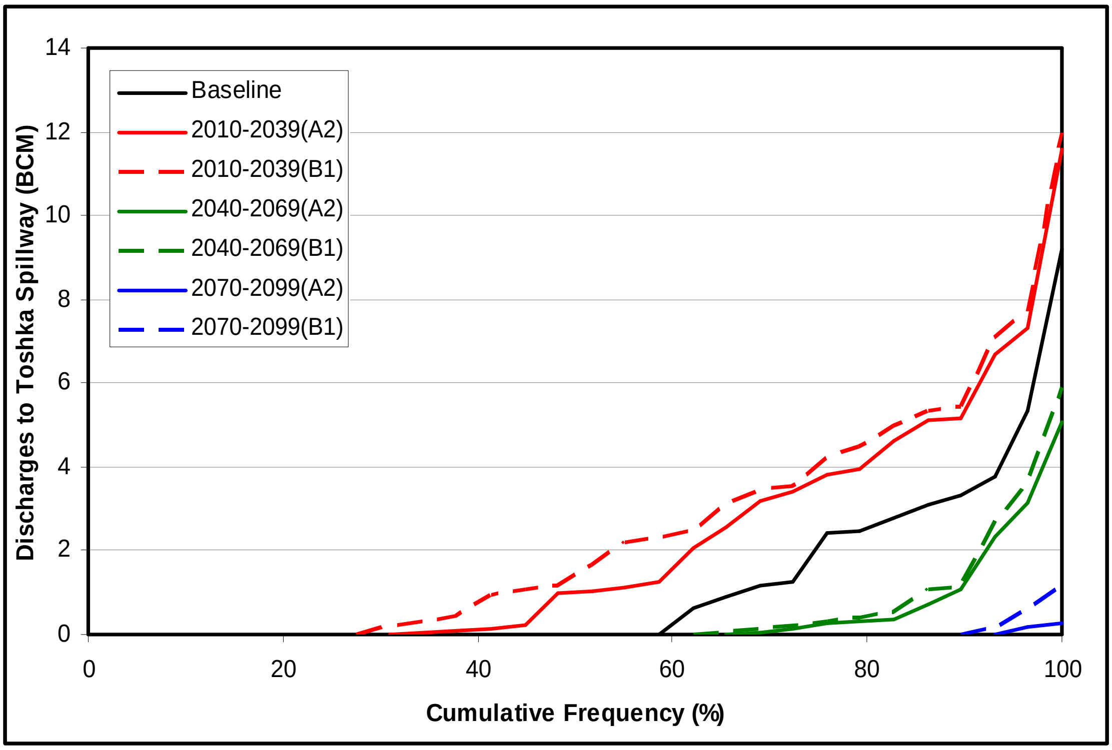


Figure A.2(h). Annual Toshka spillway discharges frequency curve for scenario II.

A.3 DEVOLOPMENT SCENARIO III

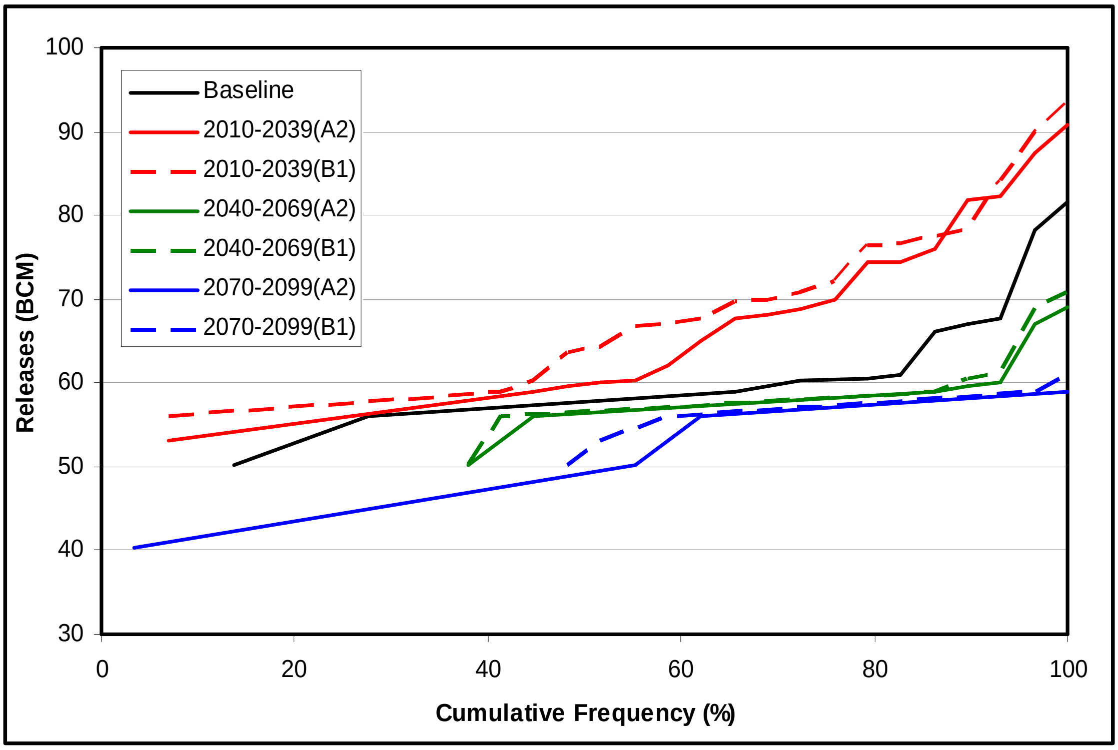


Figure A.3(a). Frequency curve of annual withdrawal from the AHDR for scenario III.

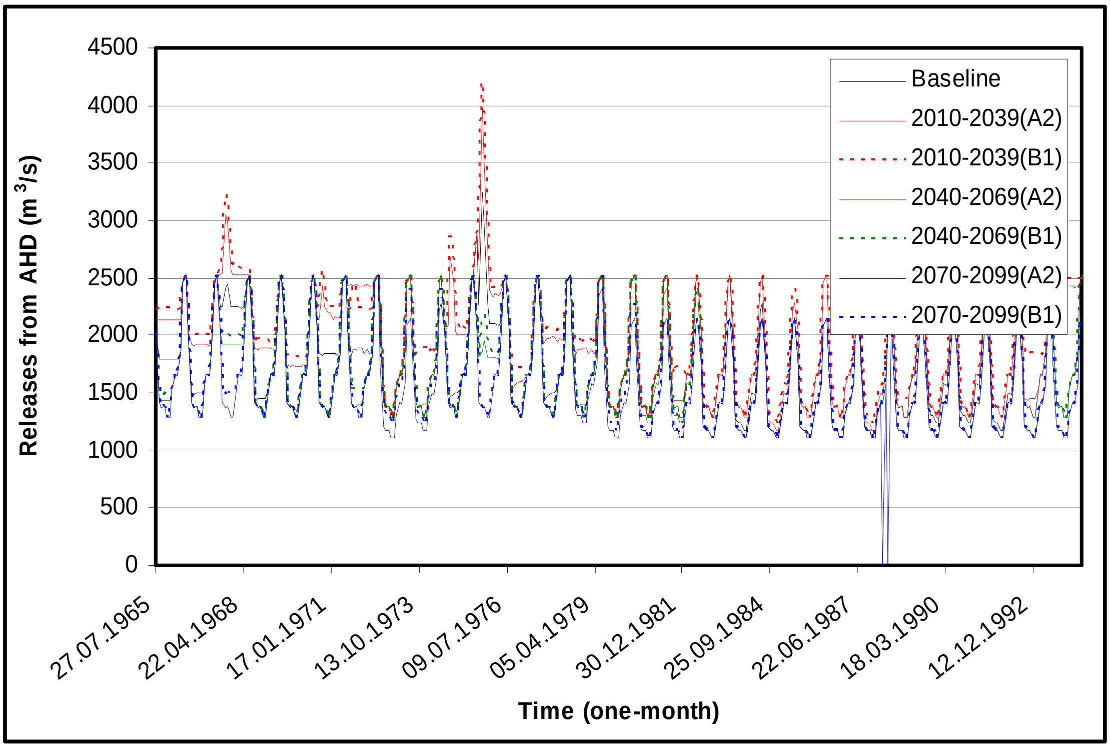


Figure A.3(b). Monthly releases from the AHD for scenario III.

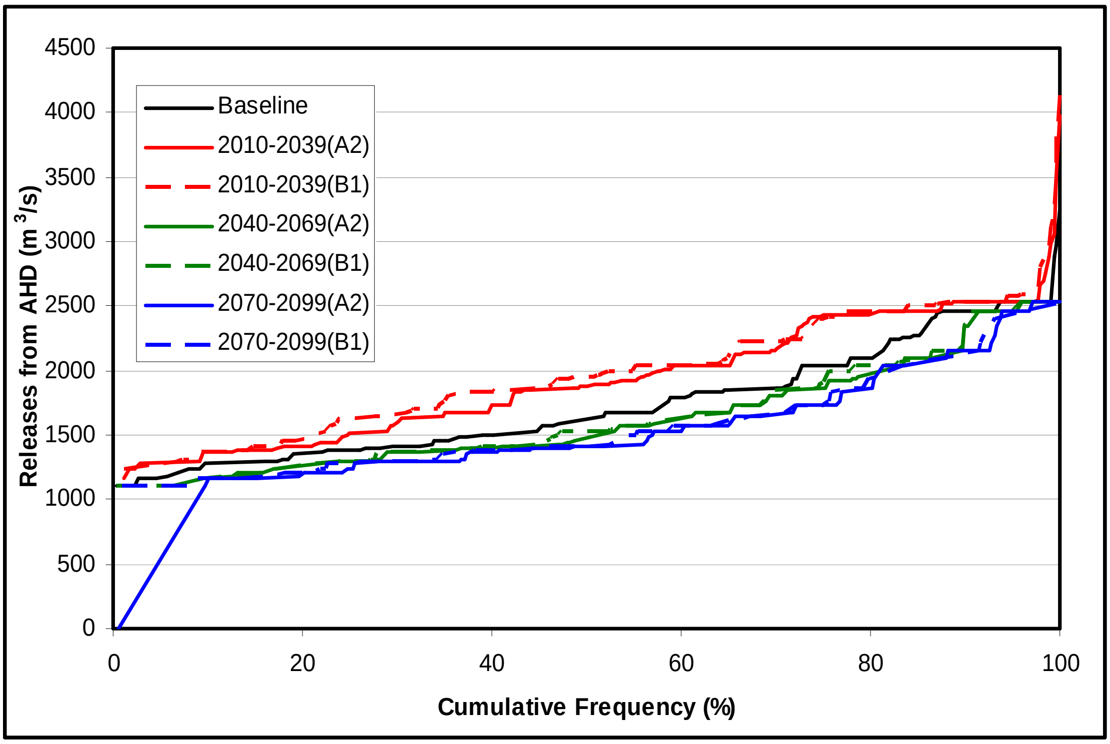


Figure A.3(c). Frequency curve of releases from the AHD for scenario III.

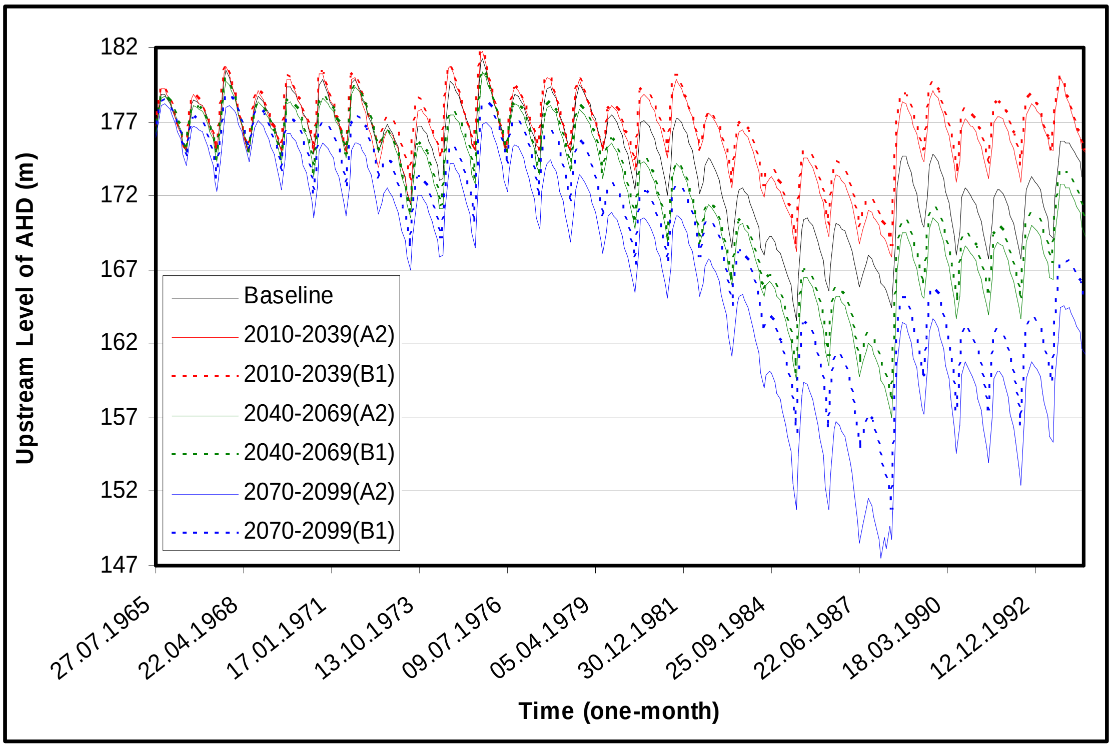


Figure A.3(d). Upstream levels of the AHD for scenario III.

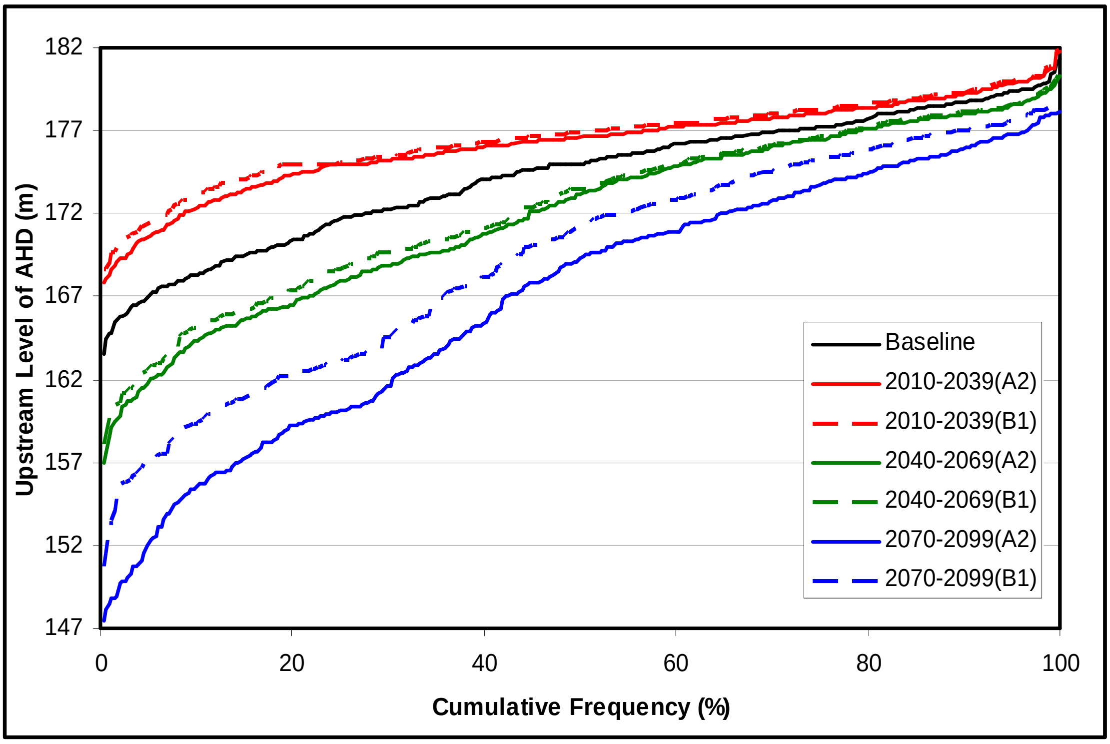


Figure A.3(e). Frequency curve of the AHD upstream levels for scenario III.

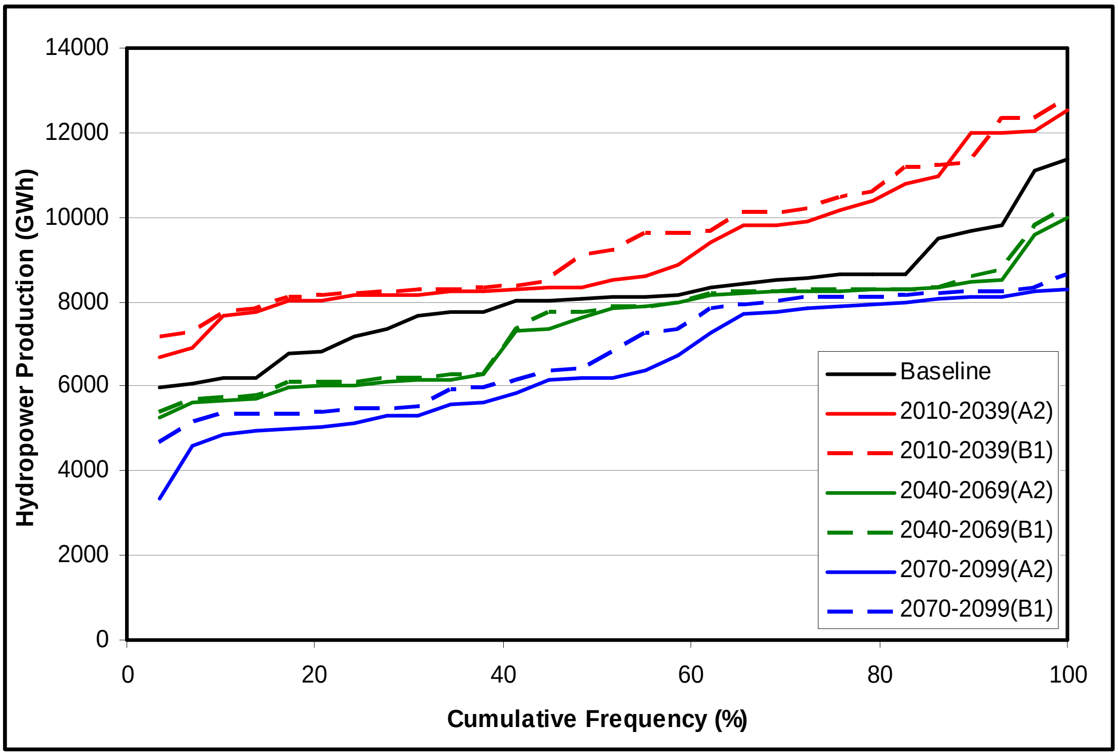


Figure A.3(f). Annual hydropower production frequency curve for scenario III.

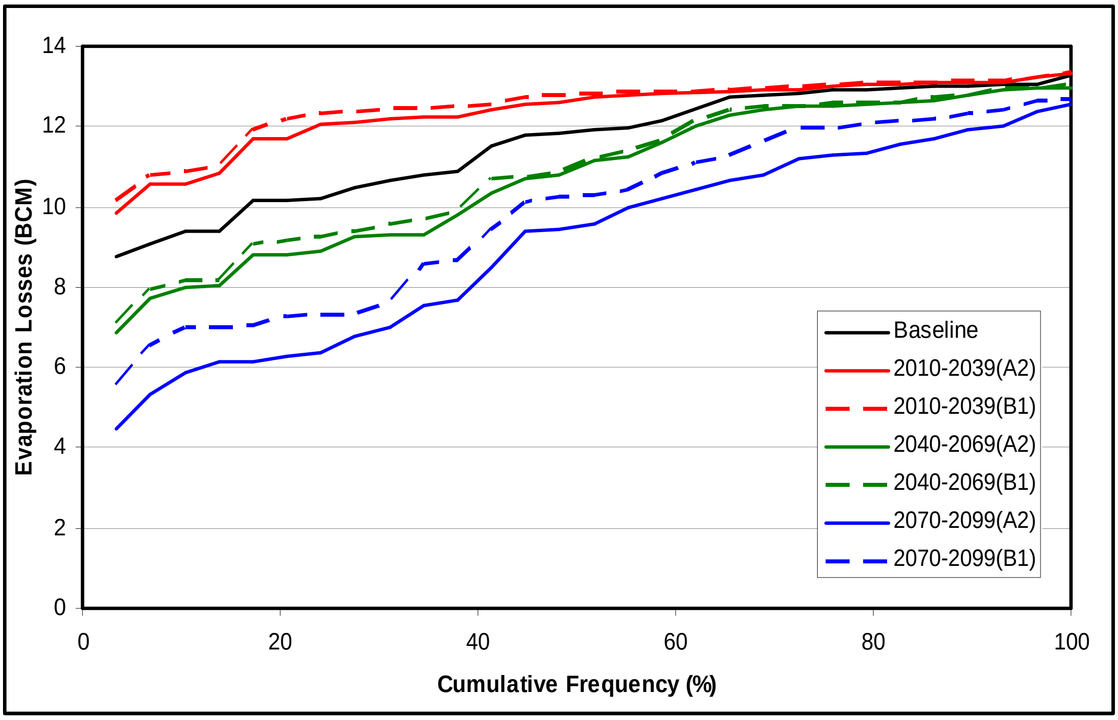


Figure A.3(g). Annual evaporation losses frequency curve for scenario III.

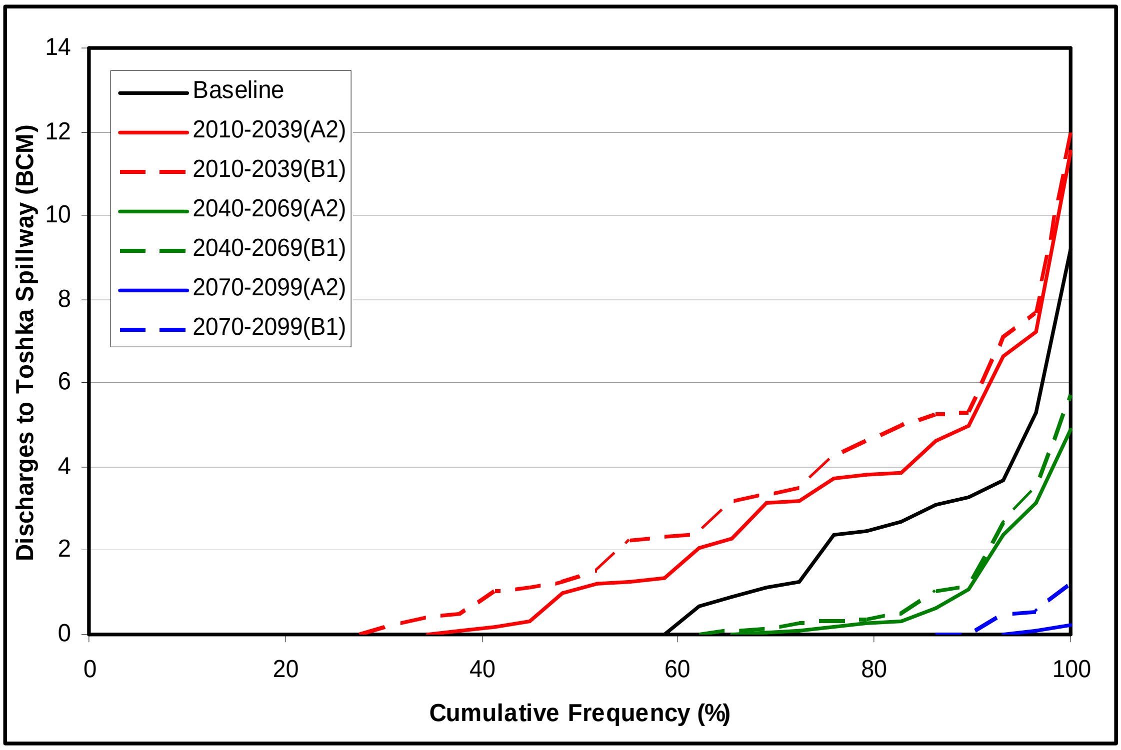


Figure A.3(h). Annual Toshka spillway discharges frequency curve for scenario III.

A.4 DEVOLOPMENT SCENARIO IV

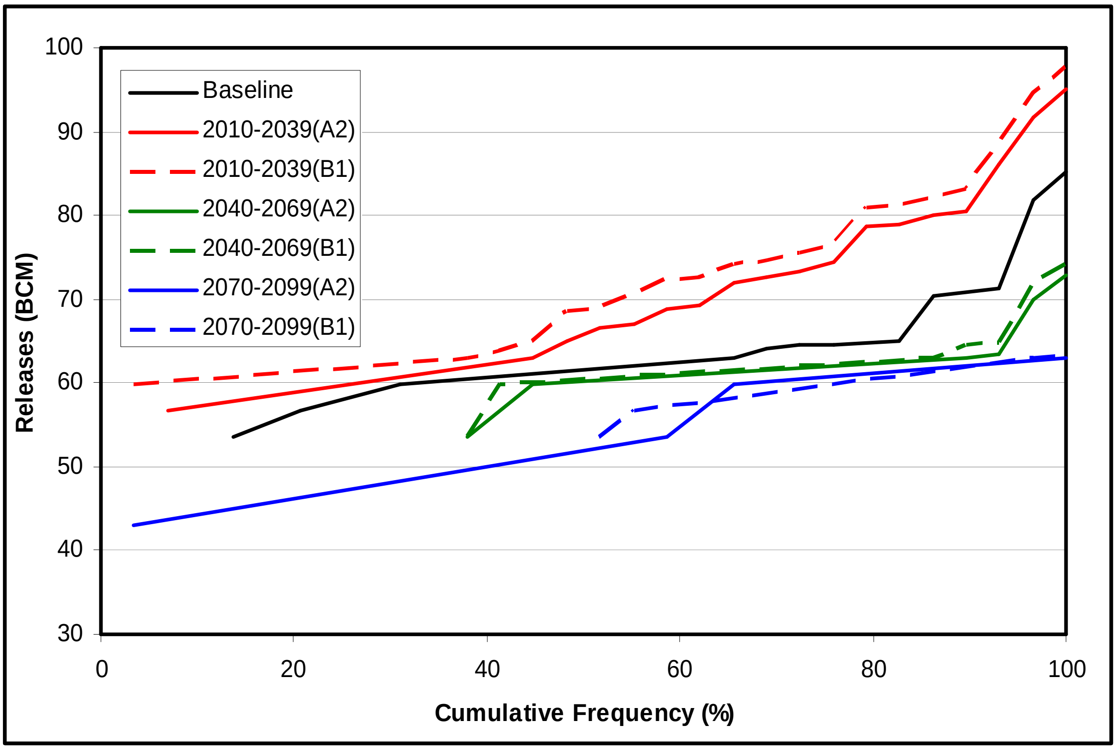


Figure A.4(a). Frequency curve of annual withdrawal from the AHDR for scenario IV.

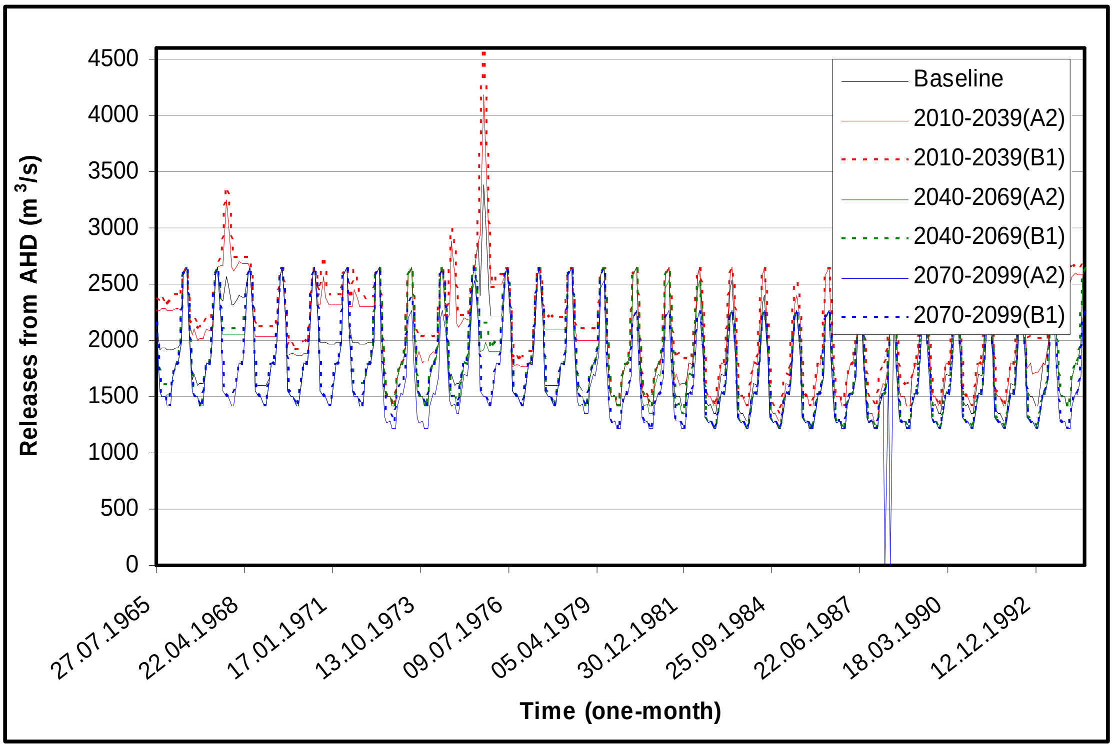


Figure A.4(b). Monthly releases from the AHD for scenario IV.

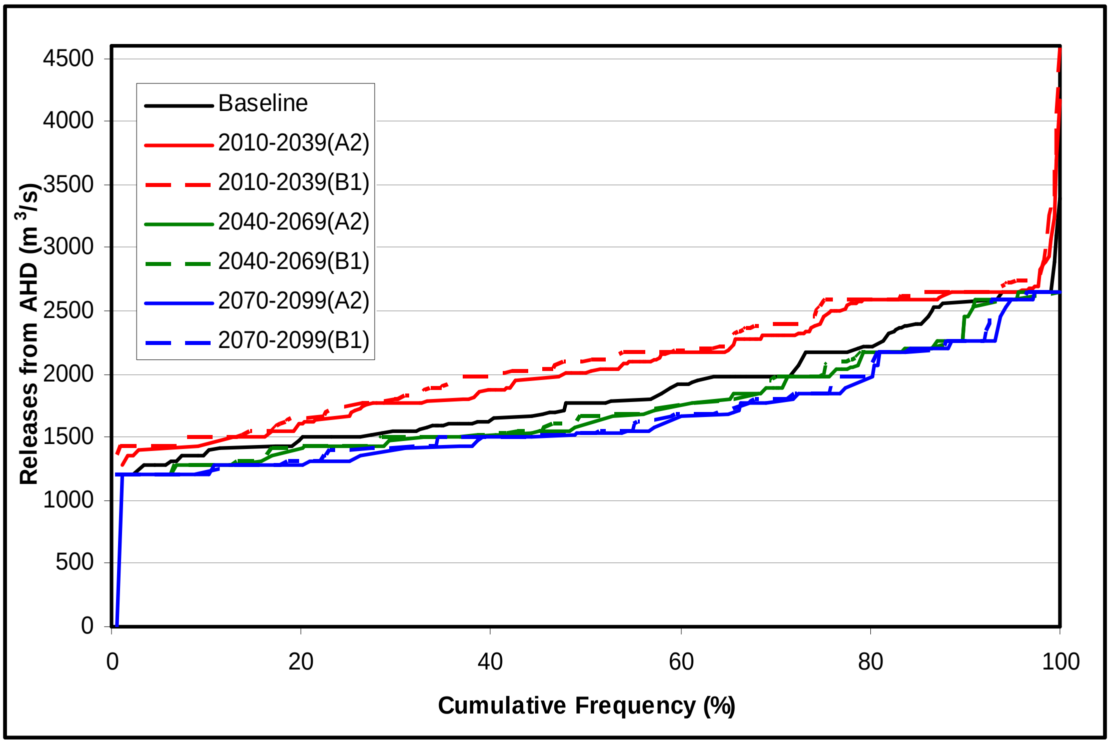


Figure A.4(c). Frequency curve of releases from the AHD for scenario IV.

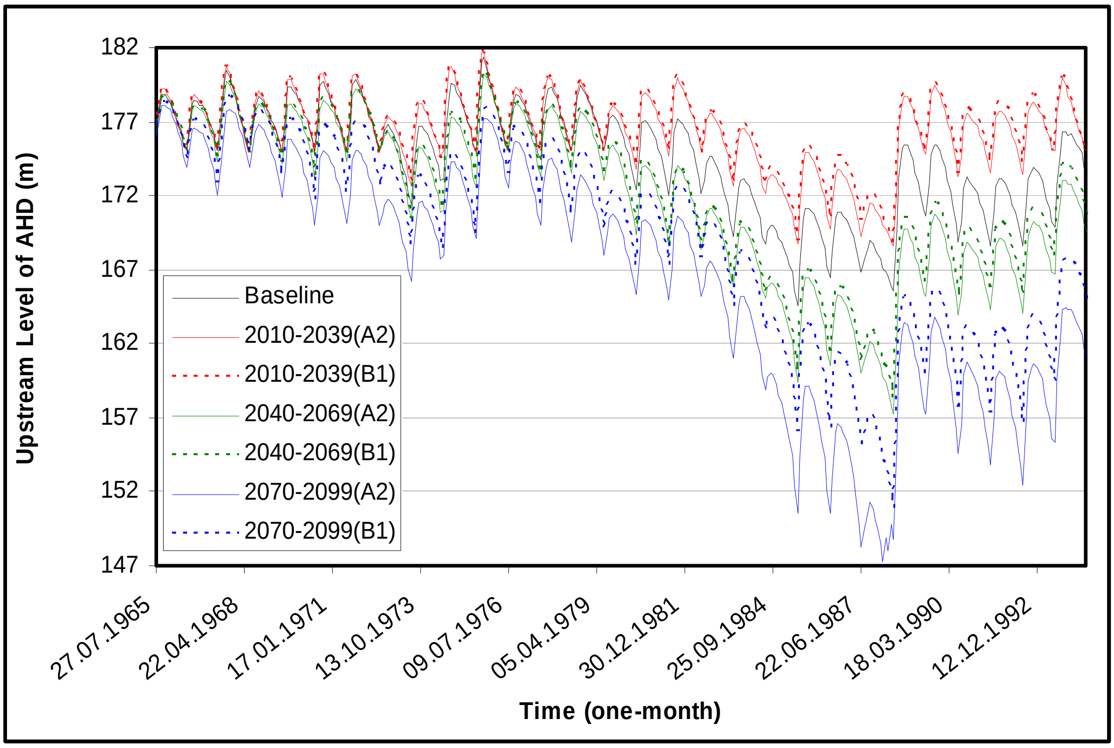


Figure A.4(d). Upstream levels of the AHD for scenario IV.

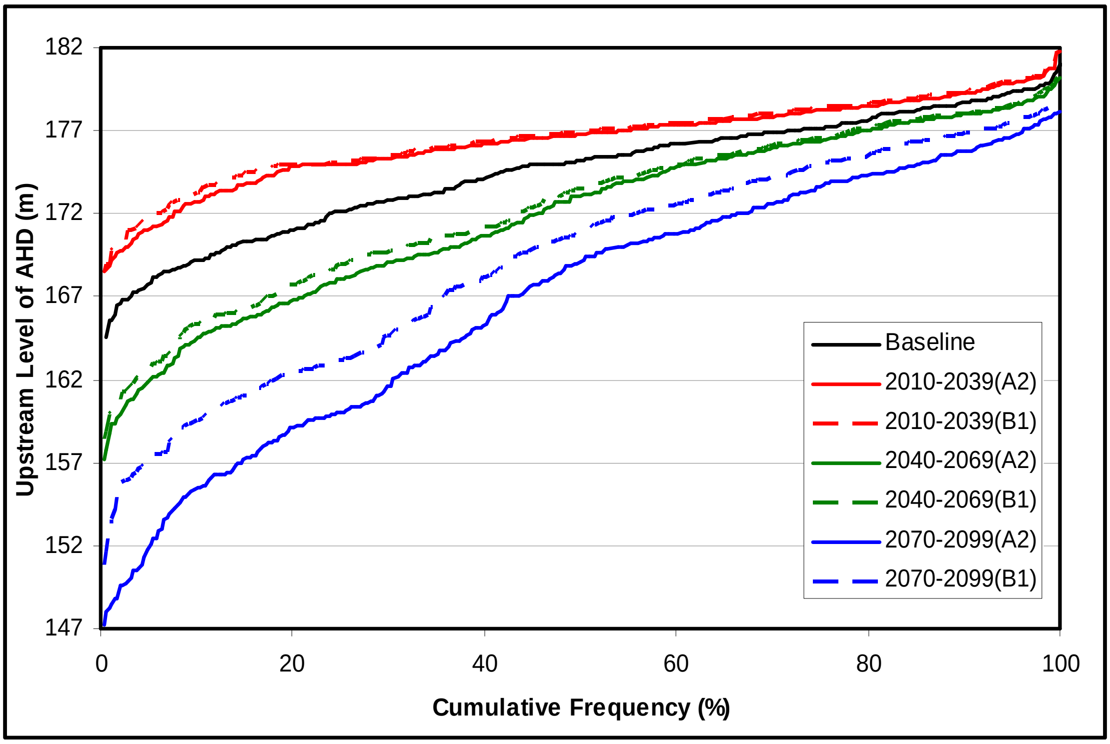


Figure A.4(e). Frequency curve of the AHD upstream levels for scenario IV.

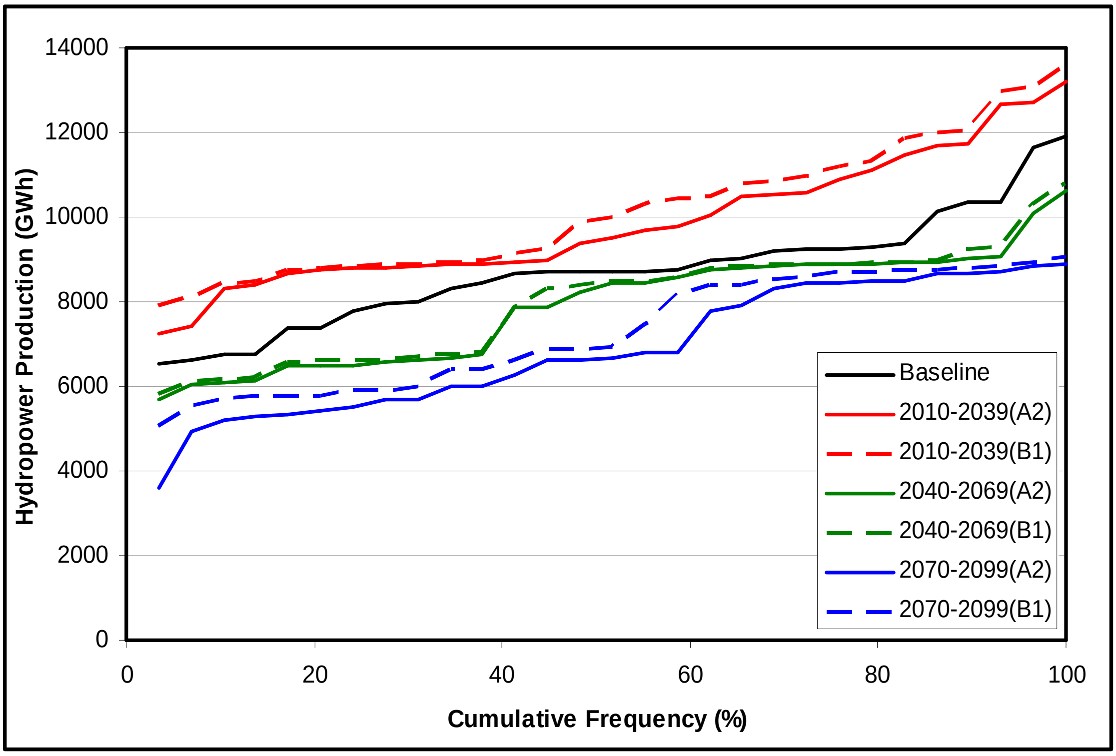


Figure A.4(f). Annual hydropower production frequency curve for scenario IV.

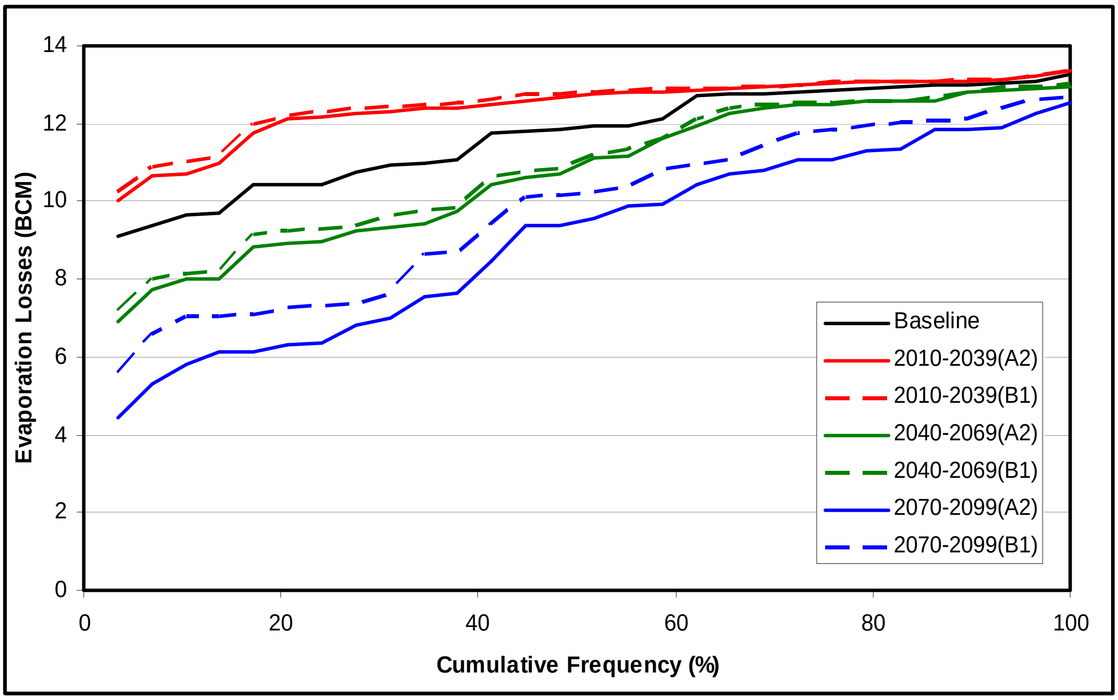


Figure A.4(g). Annual evaporation losses frequency curve for scenario IV.

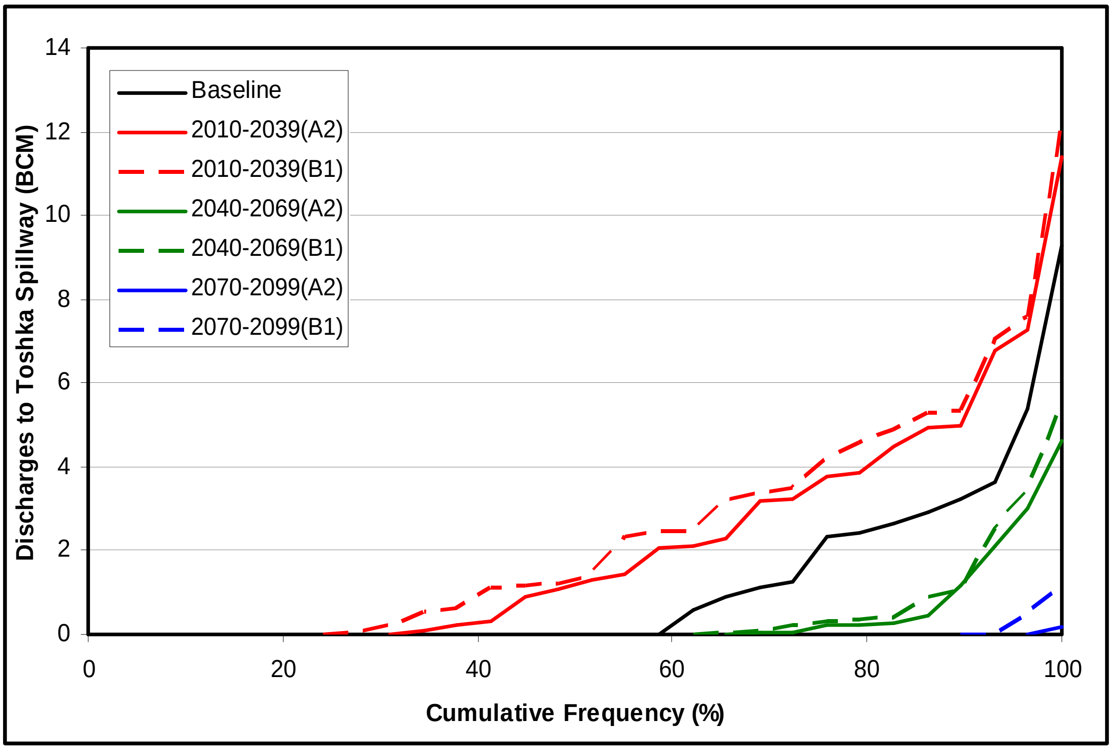


Figure A.4(h). Annual Toshka spillway discharges frequency curve for scenario IV.

APPENDIX B: EFFECT OF THE OPT POLICY ON THR OPERATIONAL PERFORMANCE FOR THE AHDR

B.1 DEVELOPMENT SCENARIO I

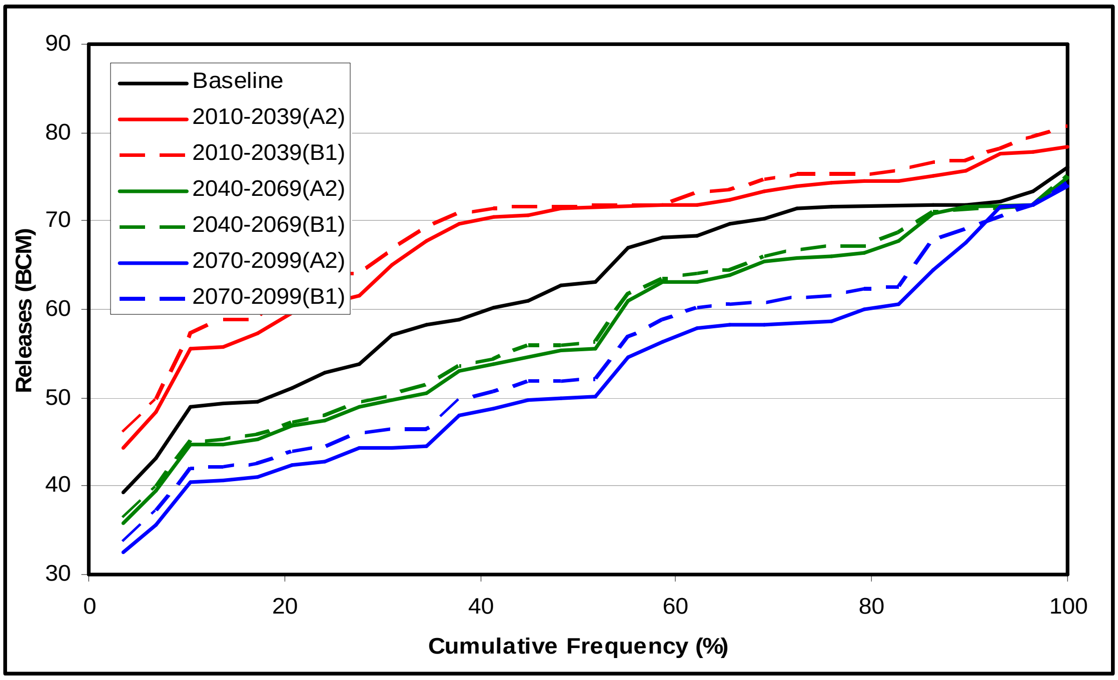


Figure B.1(a). Frequency curve of annual withdrawal from the AHDR for scenario I under the OPT policy.

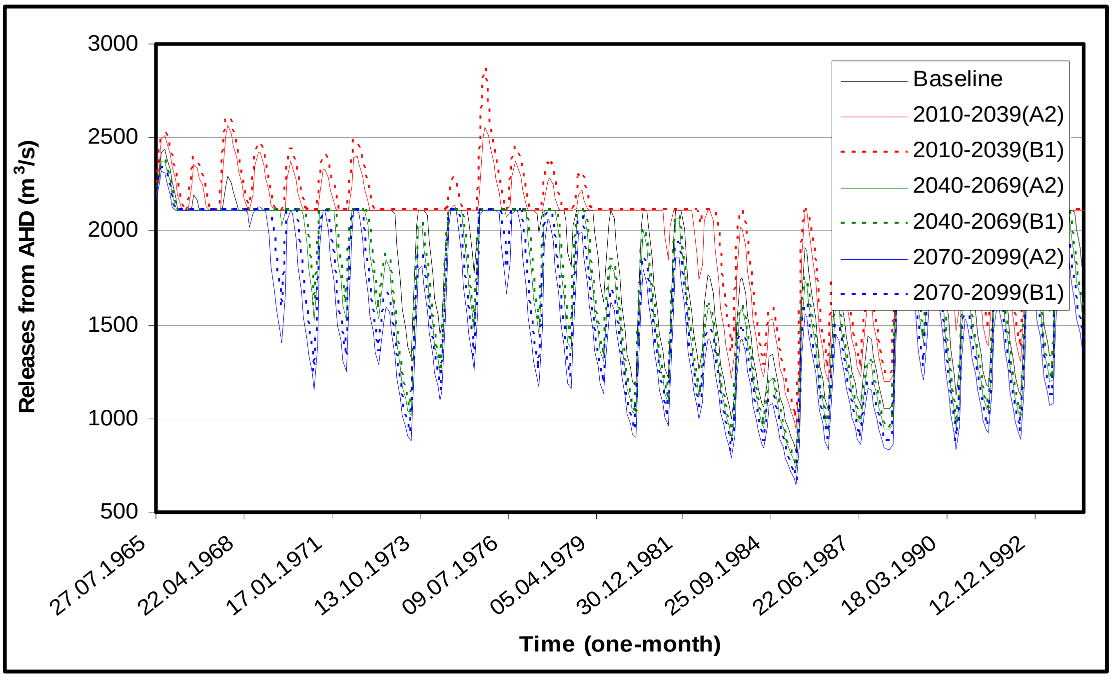


Figure B.1(b). Monthly releases from the AHD for scenario I under the OPT policy.

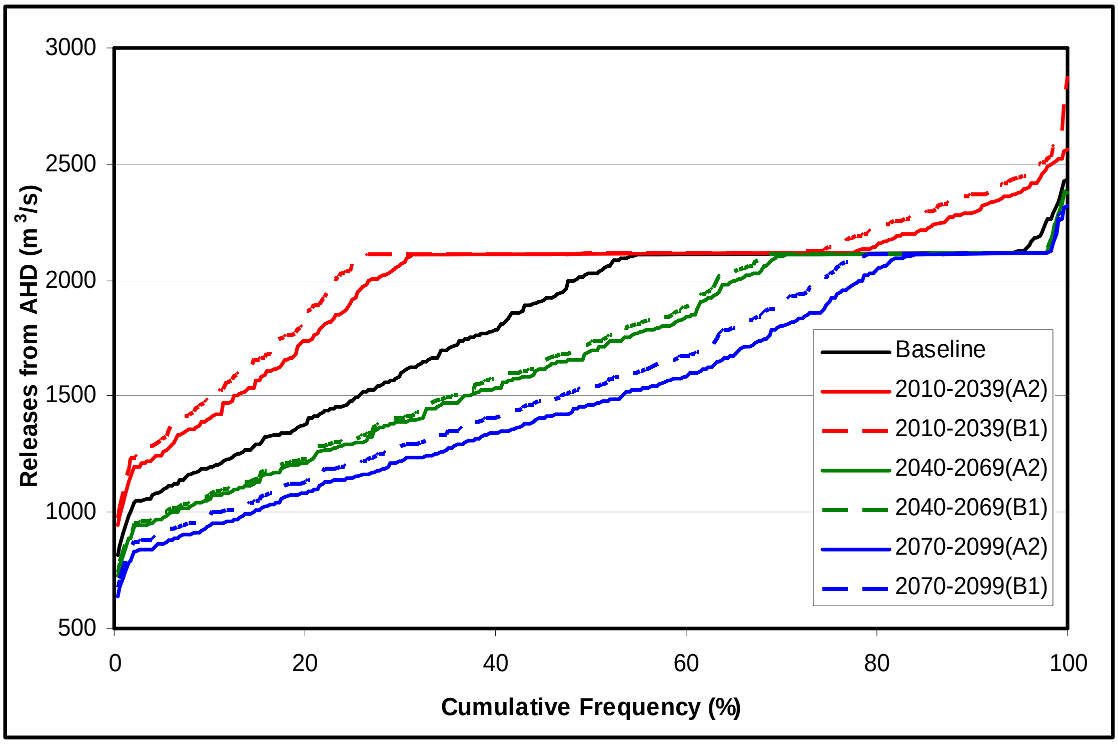


Figure B.1(c). Frequency curve of releases from the AHD for scenario I under the OPT policy.

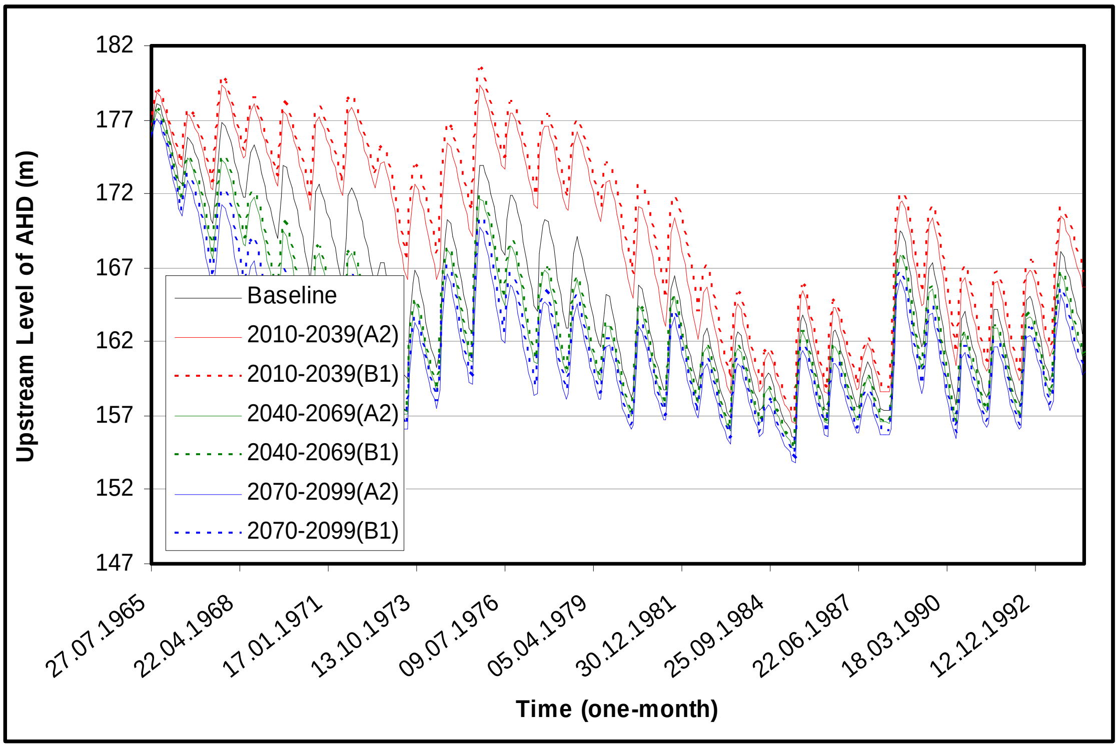


Figure B.1(d). Upstream levels of the AHD for scenario I under the OPT policy.

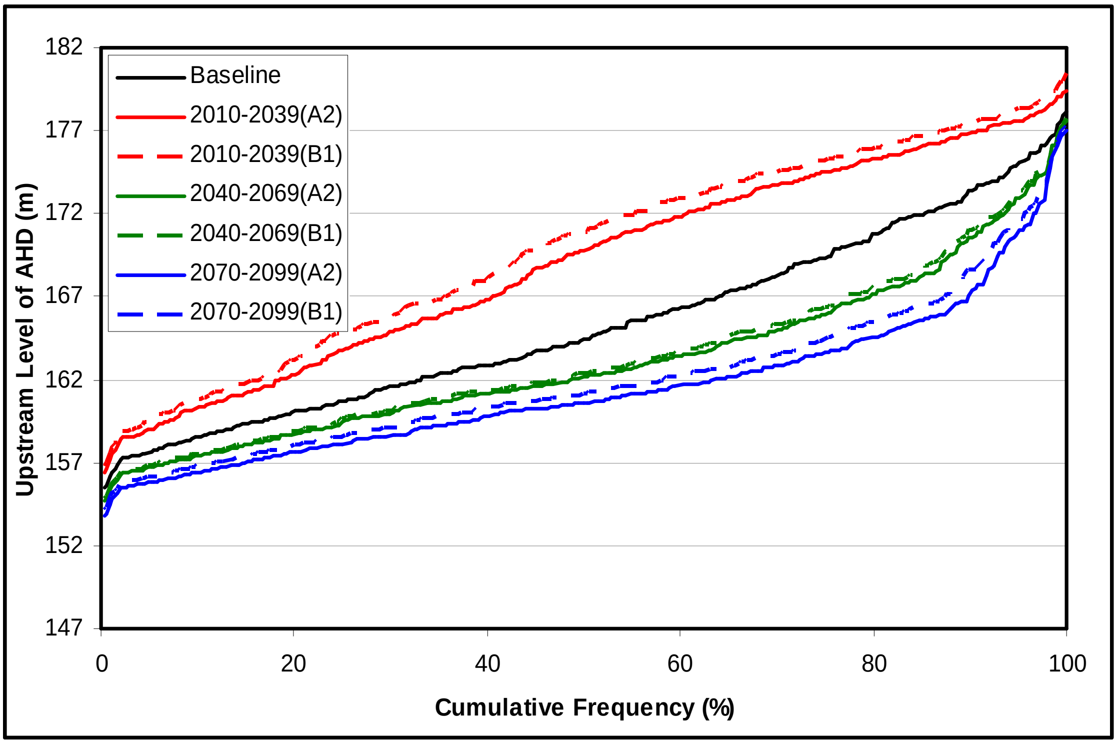


Figure B.1(e). Frequency curve of the AHD Upstream levels for scenario I under the OPT policy.

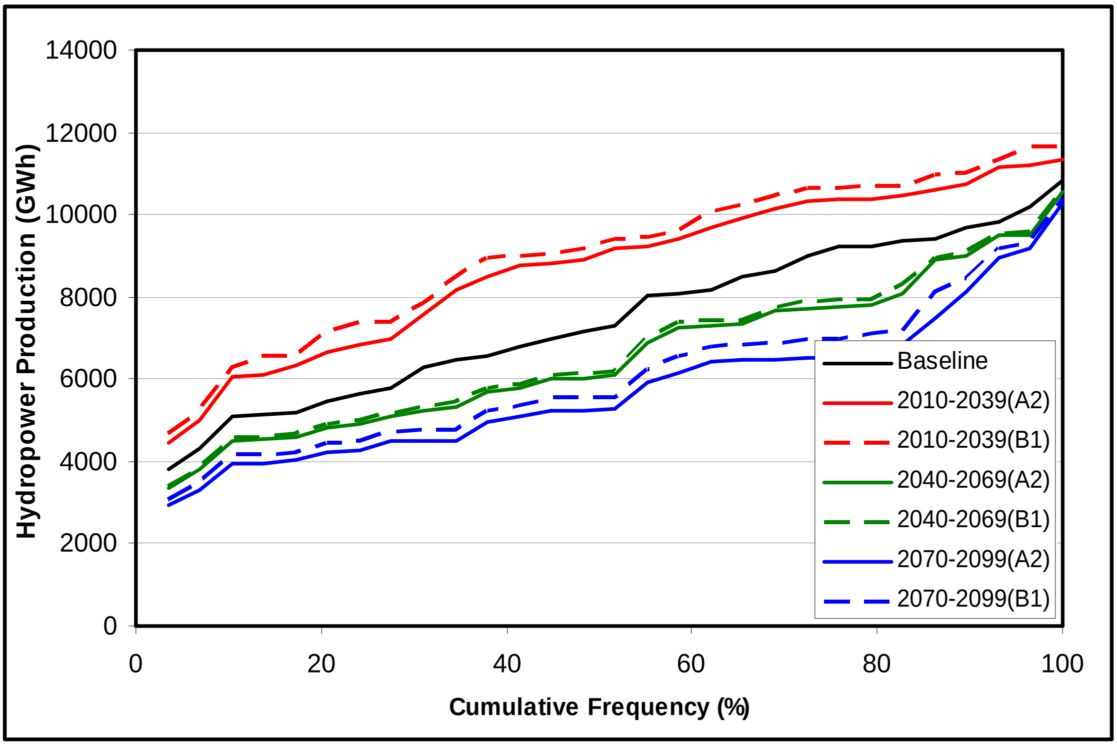


Figure B.1(f). Annual hydropower production frequency curve for scenario I under the OPT policy.

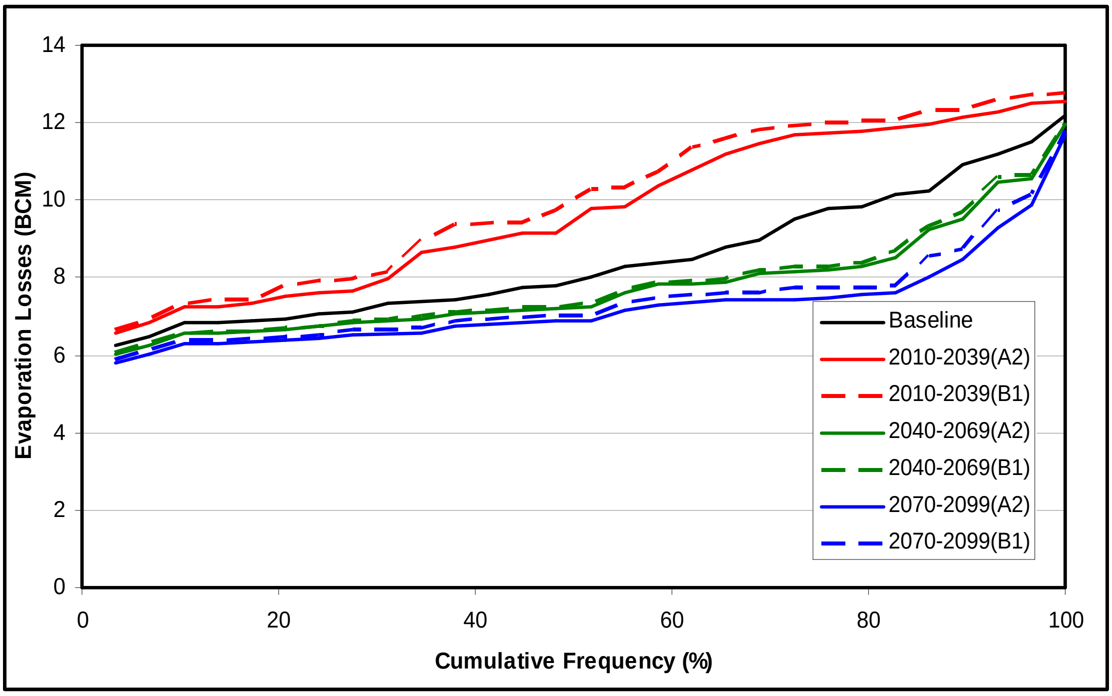


Figure B.1(g). Annual evaporation losses frequency curve for scenario I under the OPT policy.

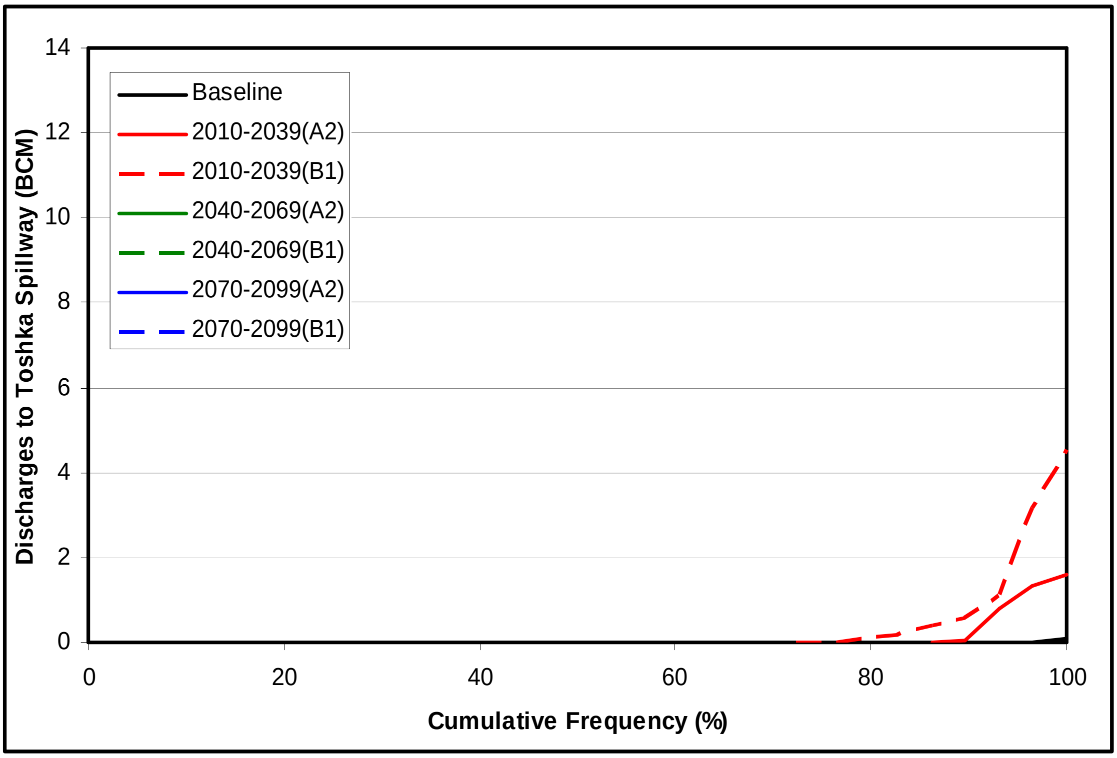


Figure B.1(h). Annual Toshka spillway discharges frequency curve for scenario I under the OPT policy.

B.2 DEVOLOPMENT SCENARIO II

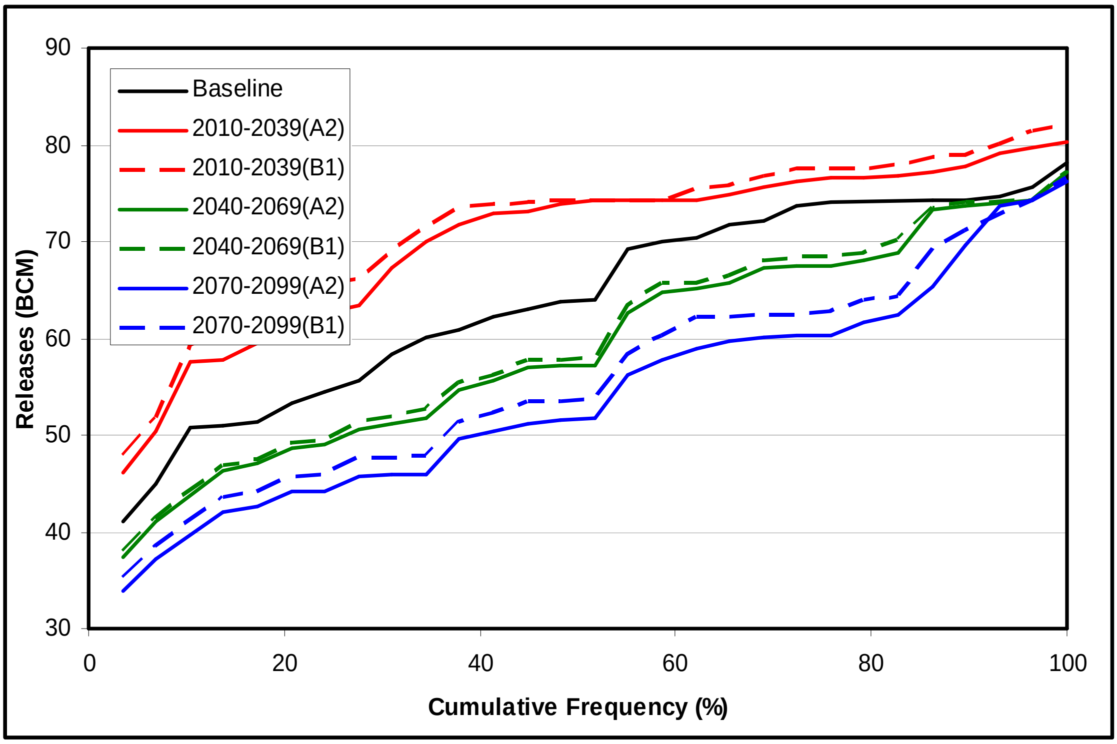


Figure B.2(a). Frequency curve of annual withdrawal from the AHDR for scenario II under the OPT policy.

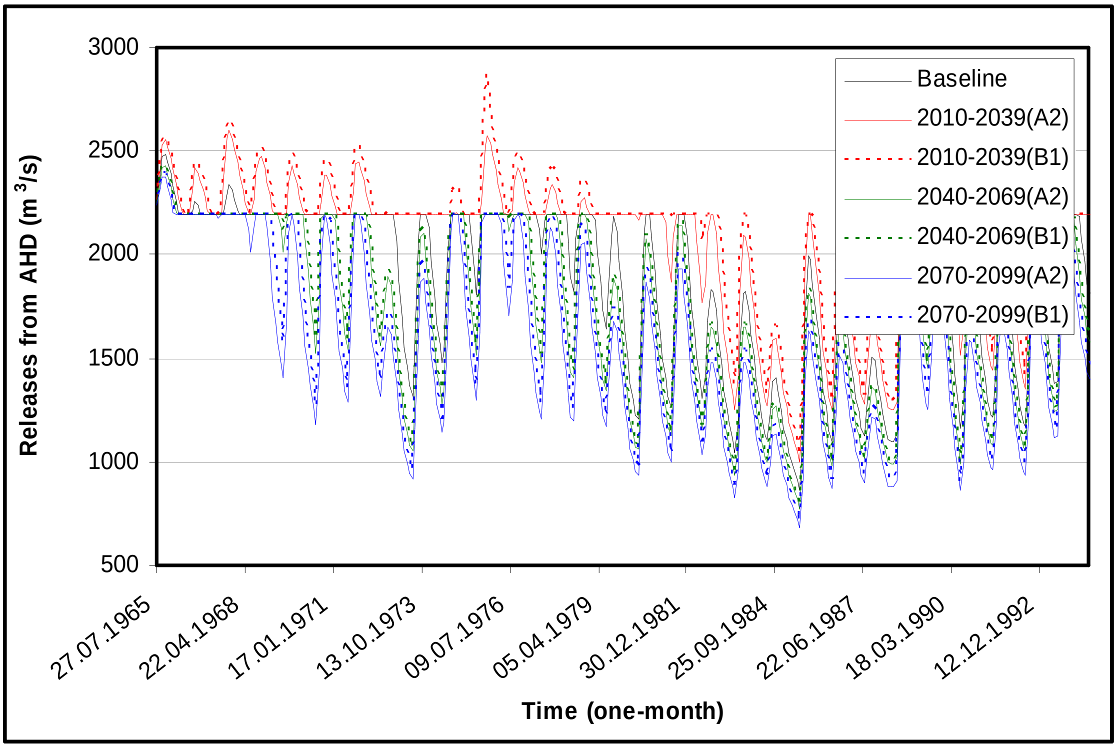


Figure B.2(b). Monthly releases from the AHD for scenario II under the OPT policy.

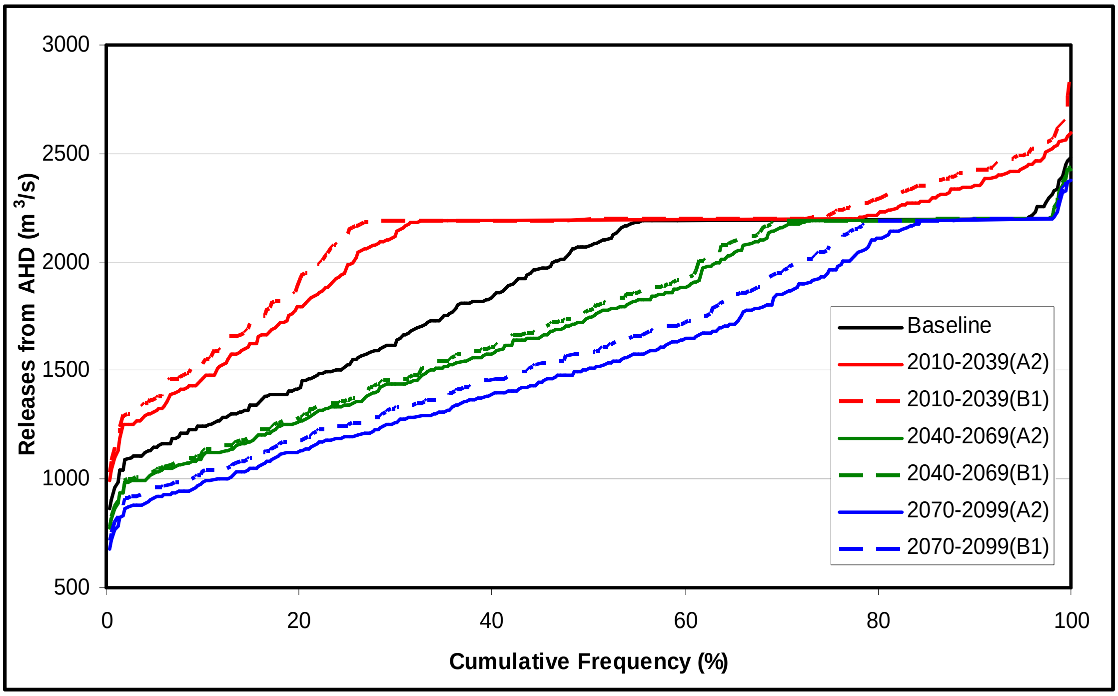


Figure B.2(c). Frequency curve of releases from the AHD for scenario II under the OPT policy.

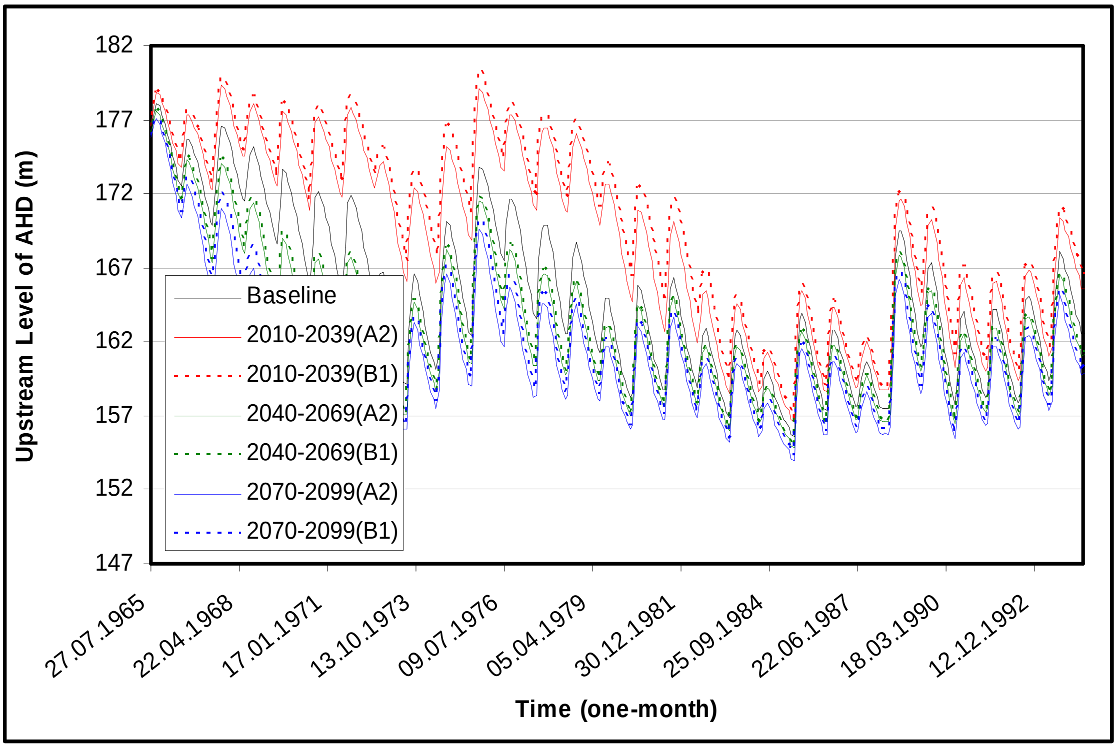


Figure B.2(d). Upstream levels of the AHD for scenario II under the OPT policy.

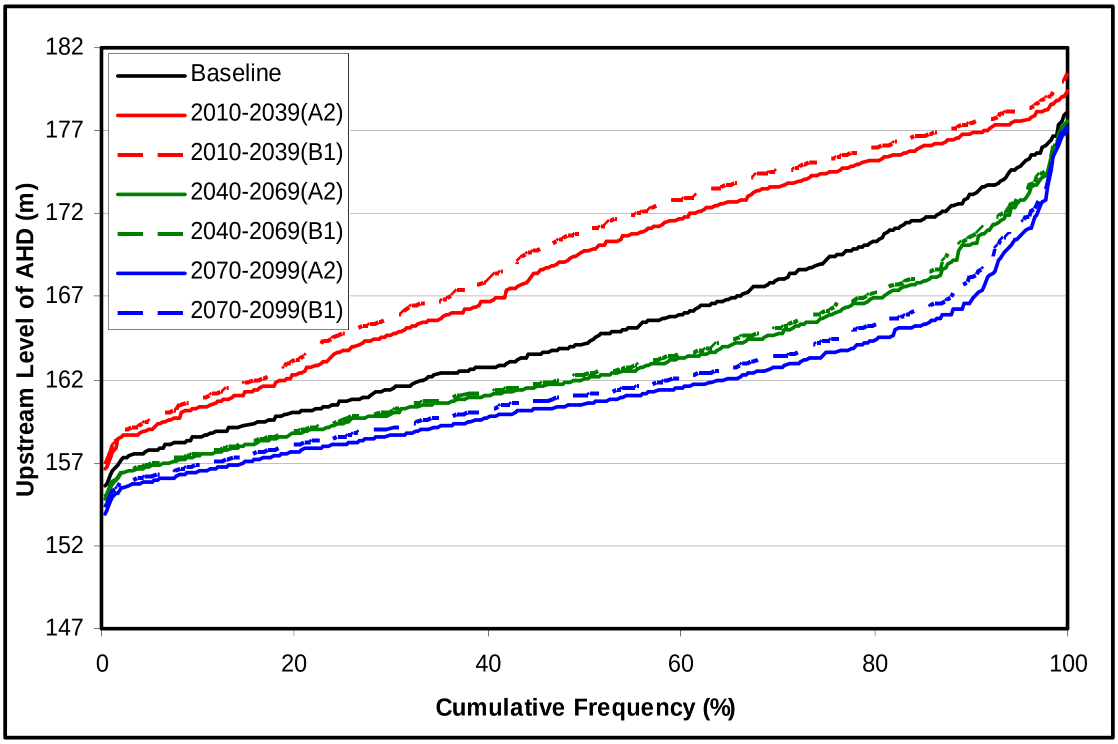


Figure B.2(e). Frequency curve of the AHD Upstream levels for scenario II under the OPT policy.

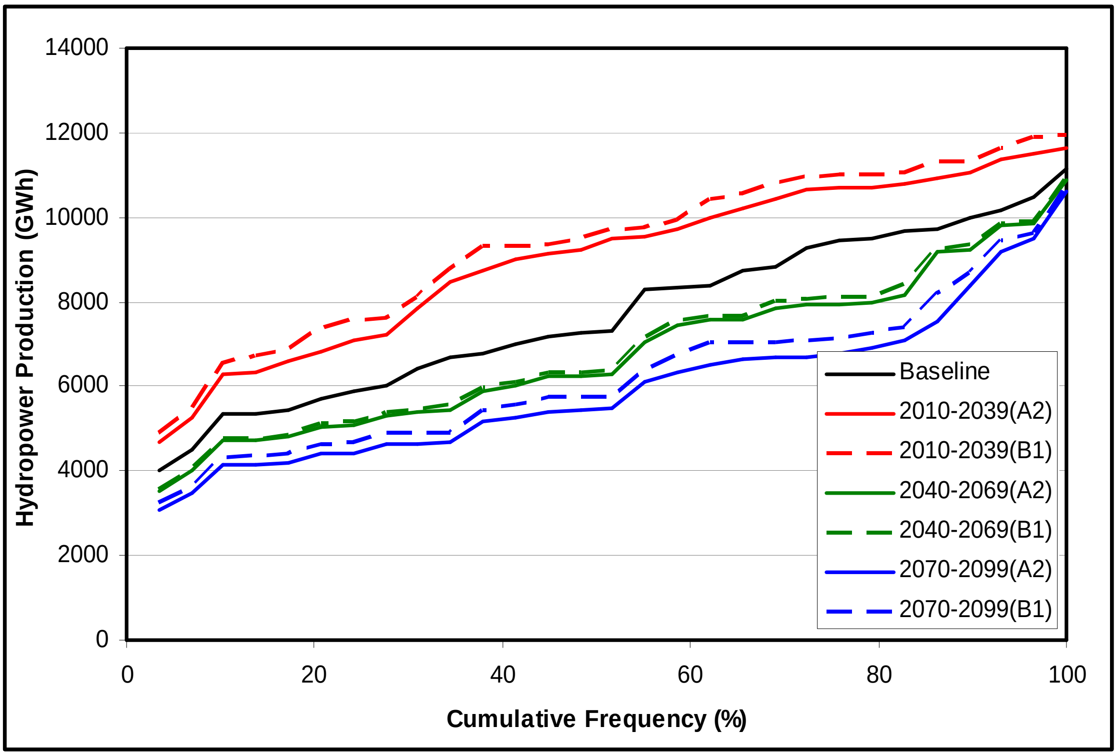


Figure B.2(f). Annual hydropower production frequency curve for scenario II under the OPT policy.

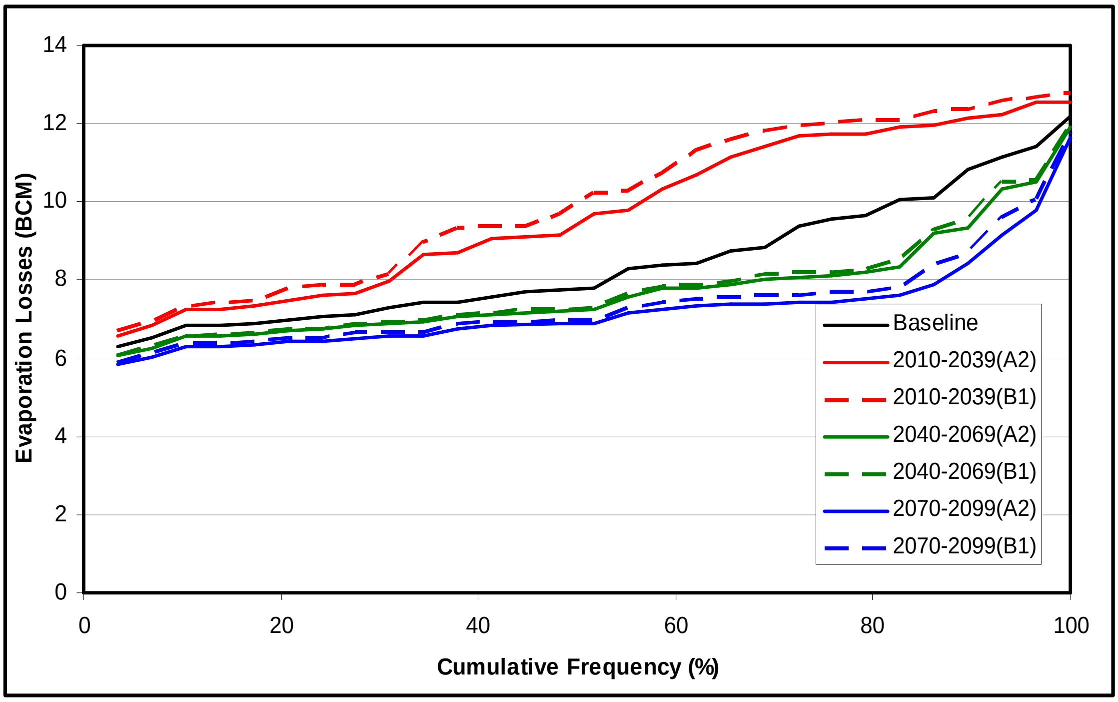


Figure B.2(g). Annual evaporation losses frequency curve for scenario II under the OPT policy.

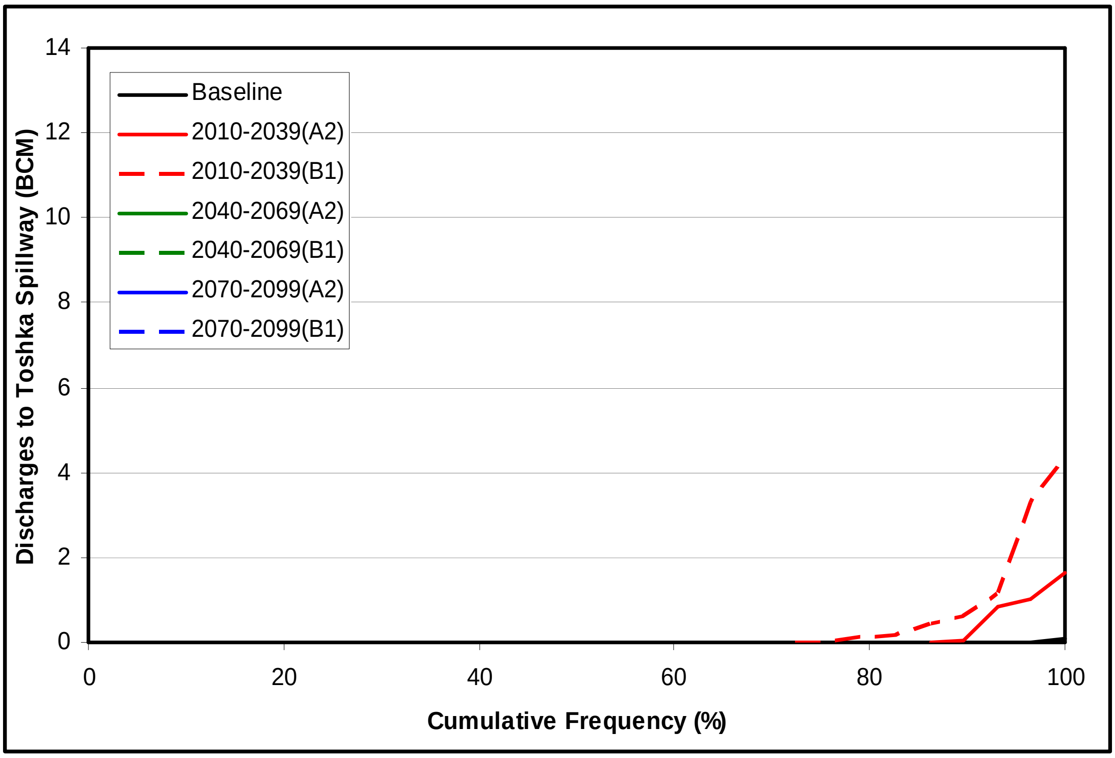


Figure B.2(h). Annual Toshka spillway discharges frequency curve for scenario II under the OPT policy.

B.3 DEVOLOPMENT SCENARIO III

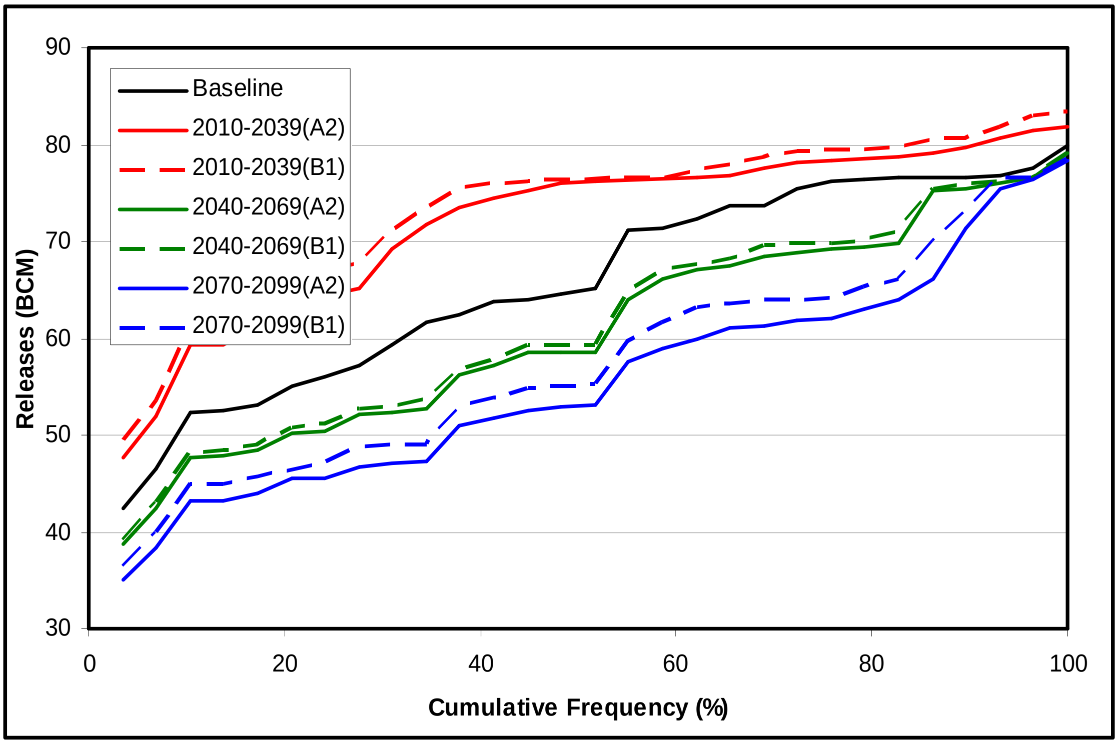


Figure B.3(a). Frequency curve of annual withdrawal from the AHDR for scenario III under the OPT policy.

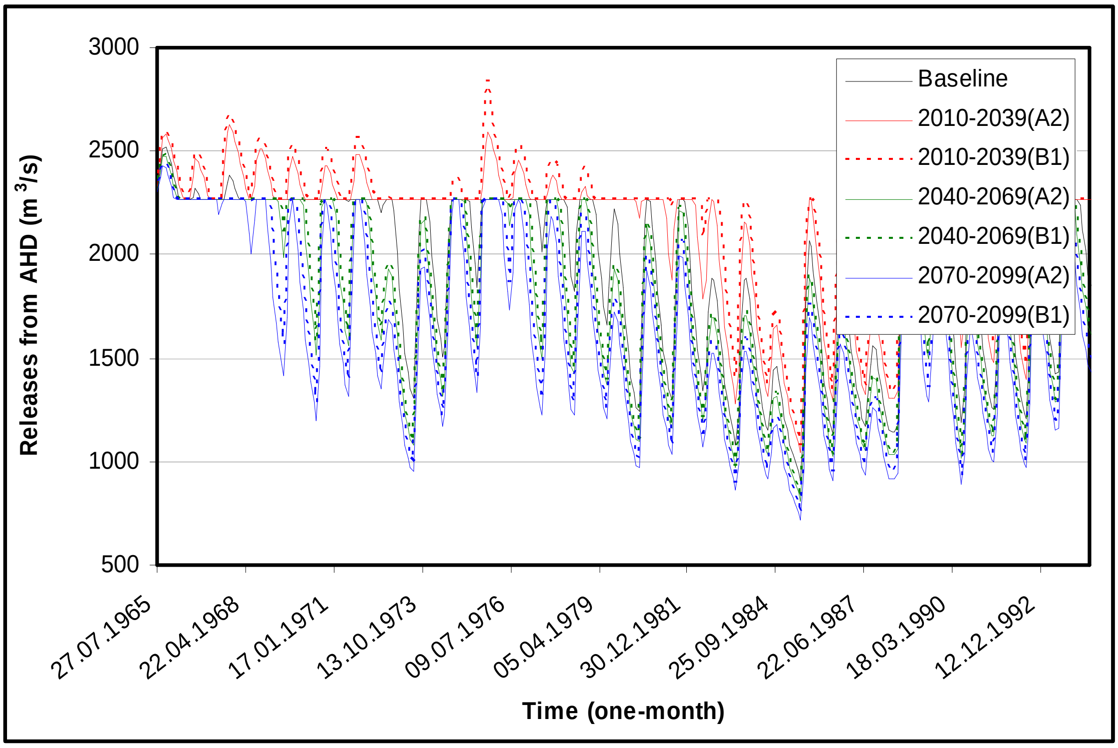


Figure B.2(b). Monthly releases from the AHD for scenario III under the OPT policy.

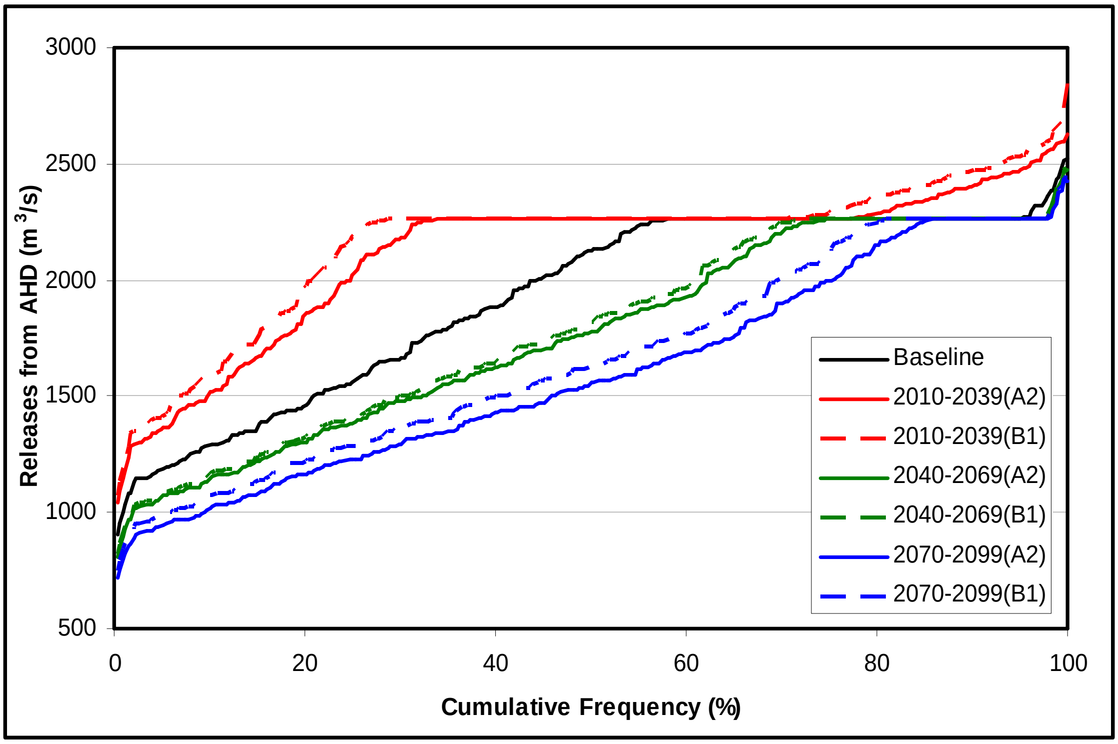


Figure B.2(c). Frequency curve of releases from the AHD for scenario III under the OPT policy.

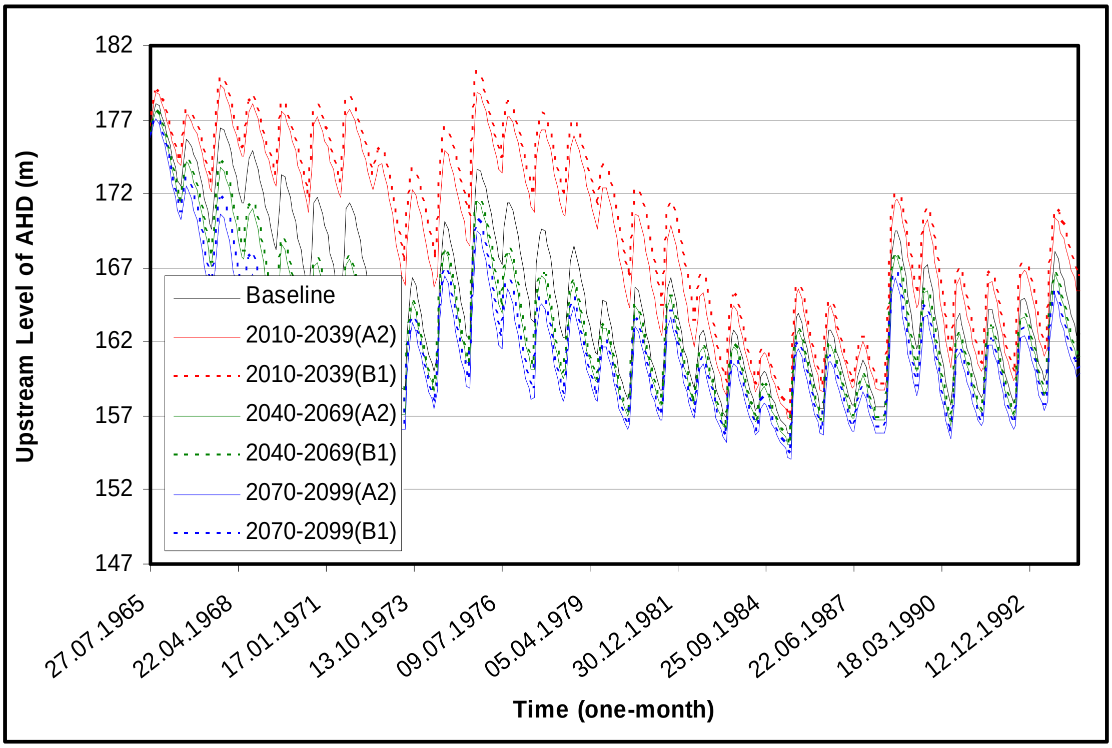


Figure B.2(d). Upstream levels of the AHD for scenario III under the OPT policy.

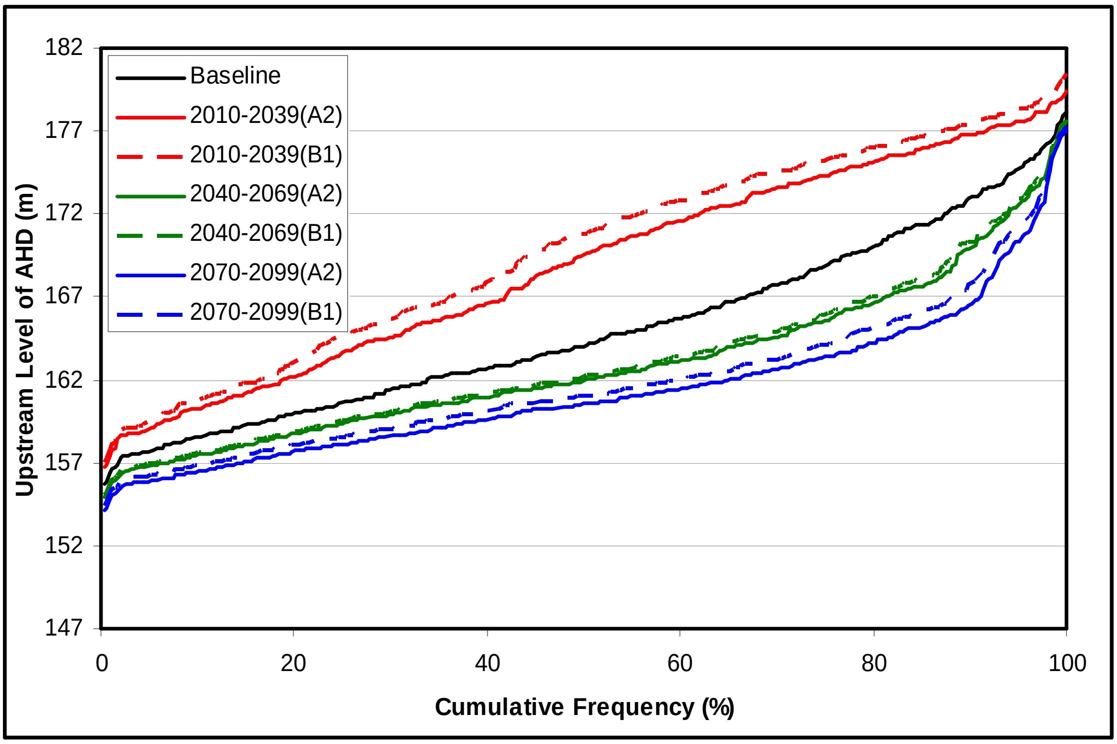


Figure B.2(e). Frequency curve of the AHD Upstream levels for scenario III under the OPT policy.

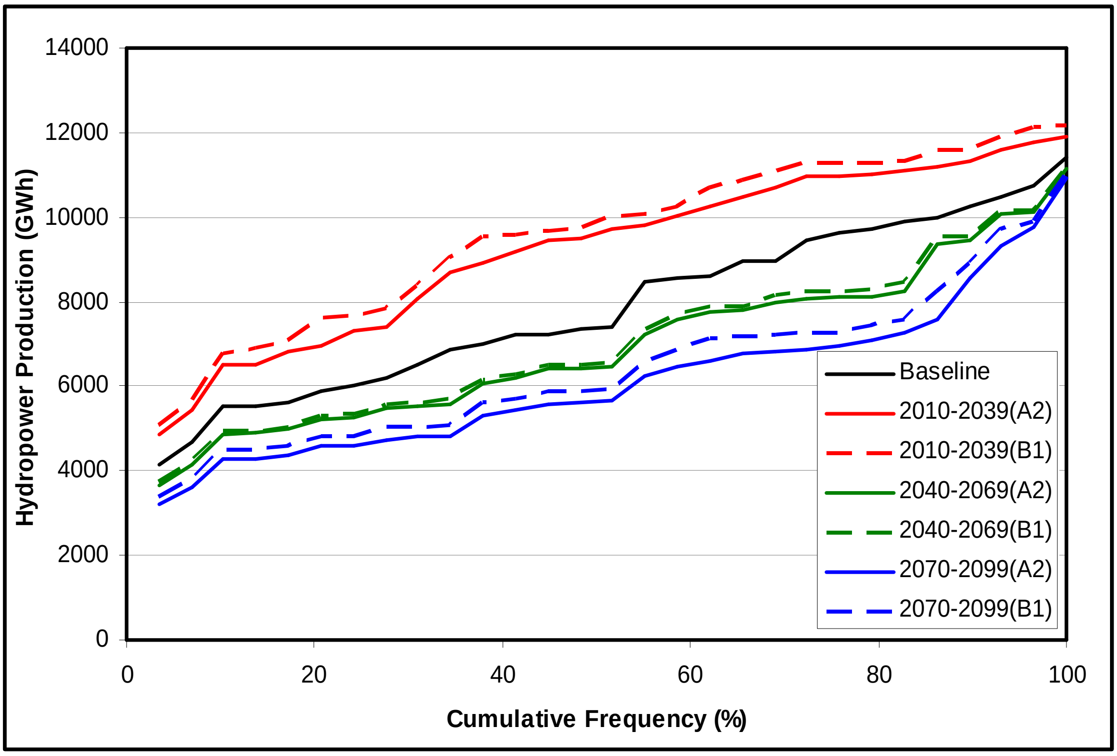


Figure B.2(f). Annual hydropower production frequency curve for scenario III under the OPT policy.

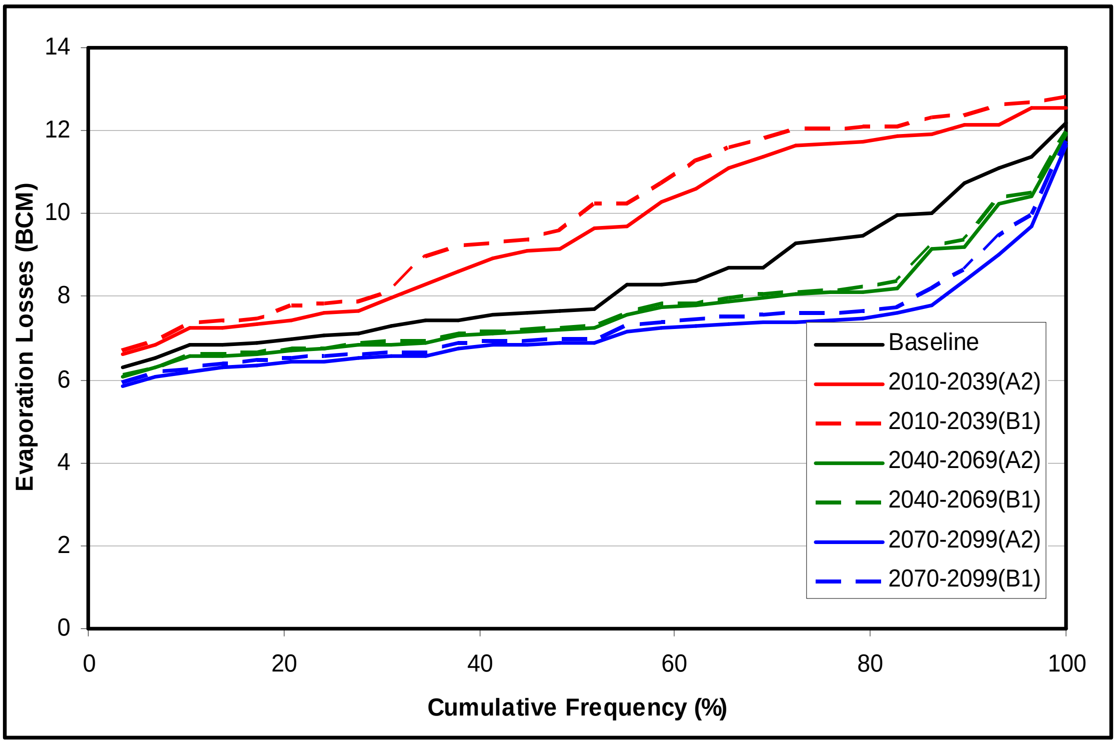


Figure B.2(g). Annual evaporation losses frequency curve for scenario III under the OPT policy.

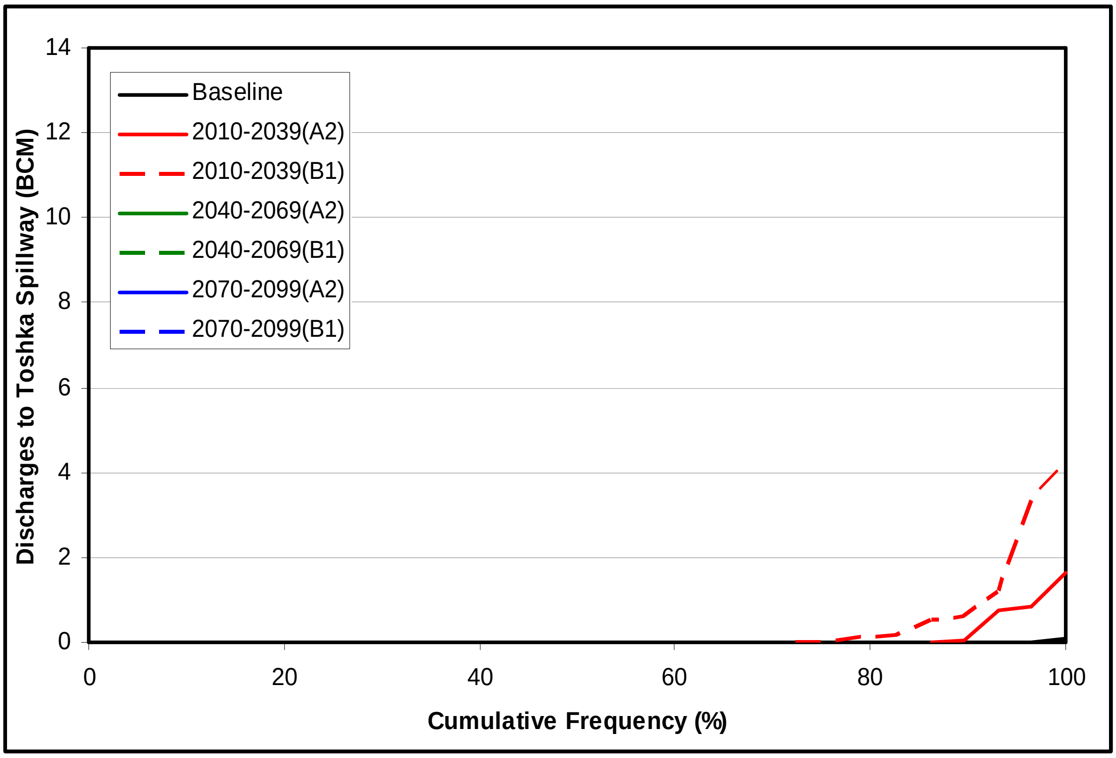


Figure B.2(h). Annual Toshka spillway discharges frequency curve for scenario III under the OPT policy.

B.4 DEVOLOPMENT SCENARIO IV

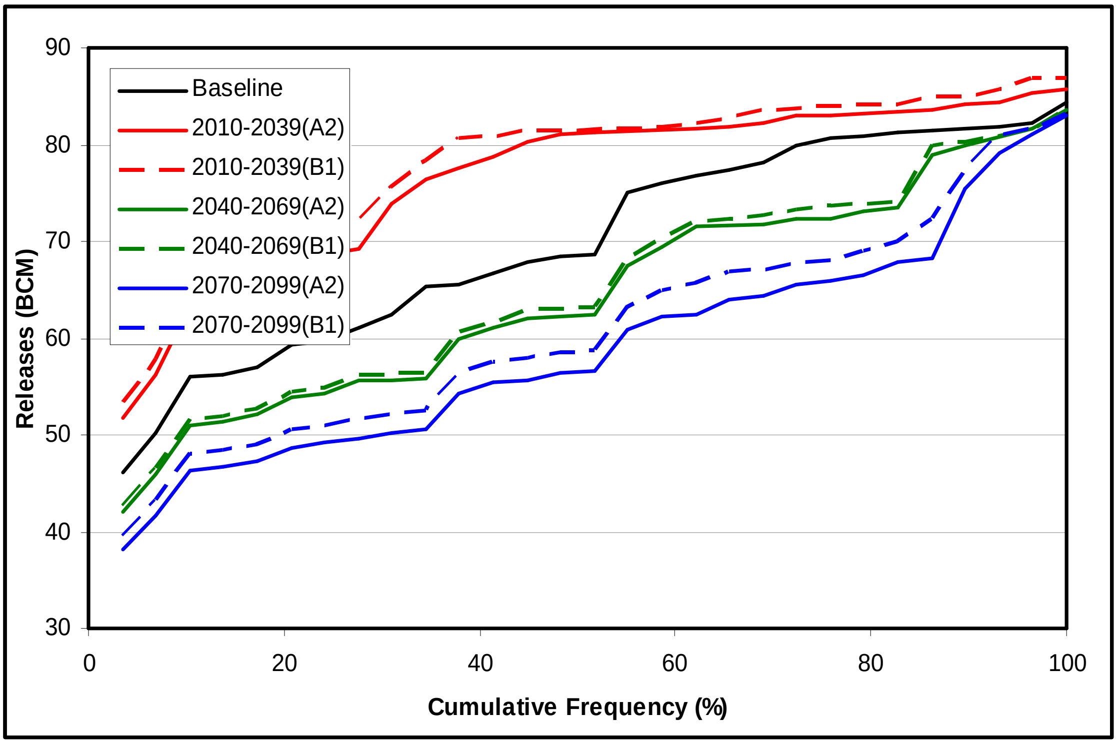


Figure B.4(a). Frequency curve of annual withdrawal from the AHDR for scenario IV under the OPT policy.

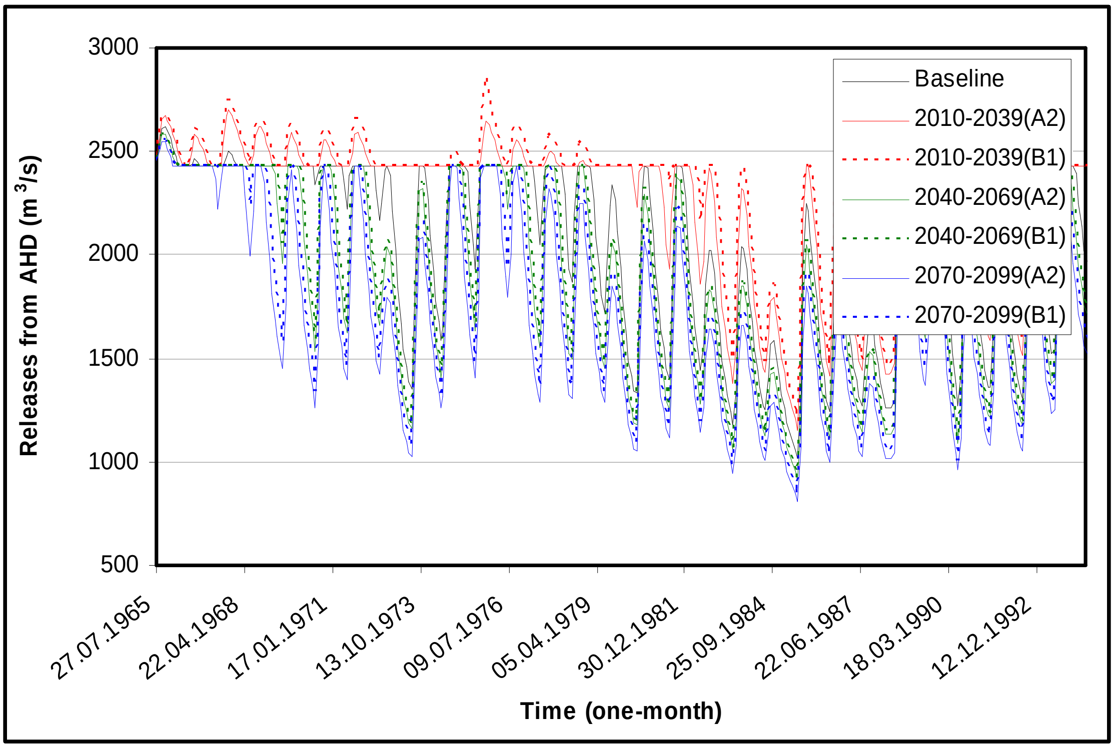


Figure B.4(b). Monthly releases from the AHD for scenario IV under the OPT policy.

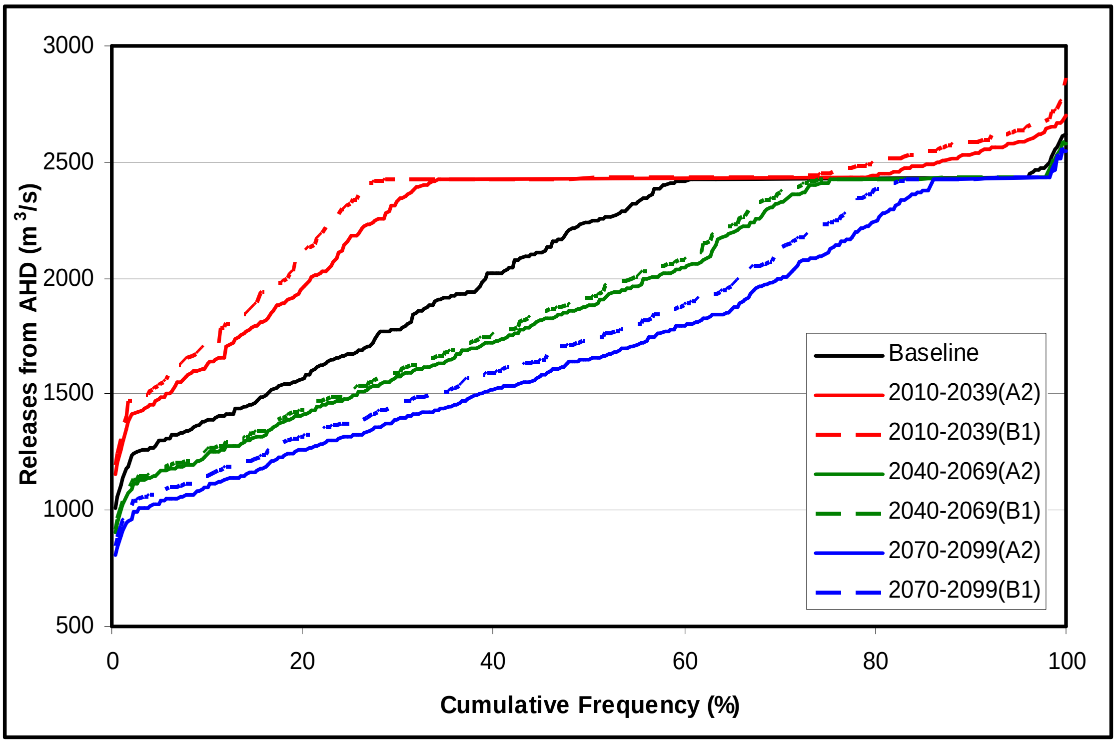


Figure B.4(c). Frequency curve of releases from the AHD for scenario IV under the OPT policy.

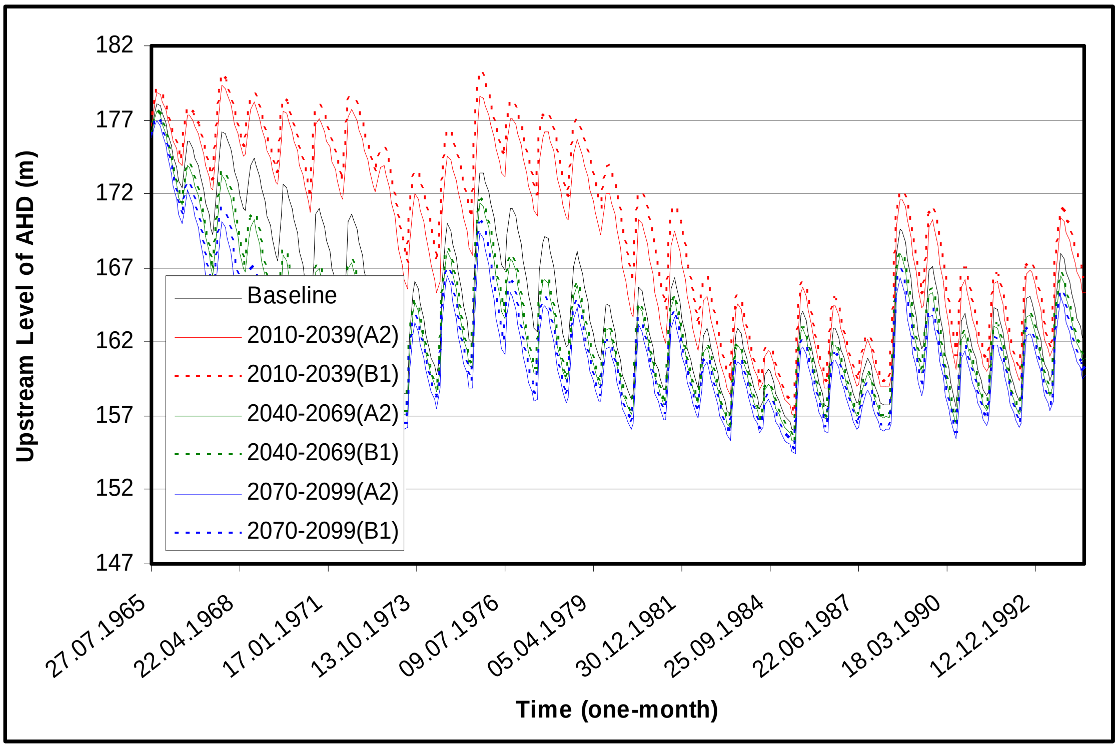


Figure B.4(d). Upstream levels of the AHD for scenario IV under the OPT policy.

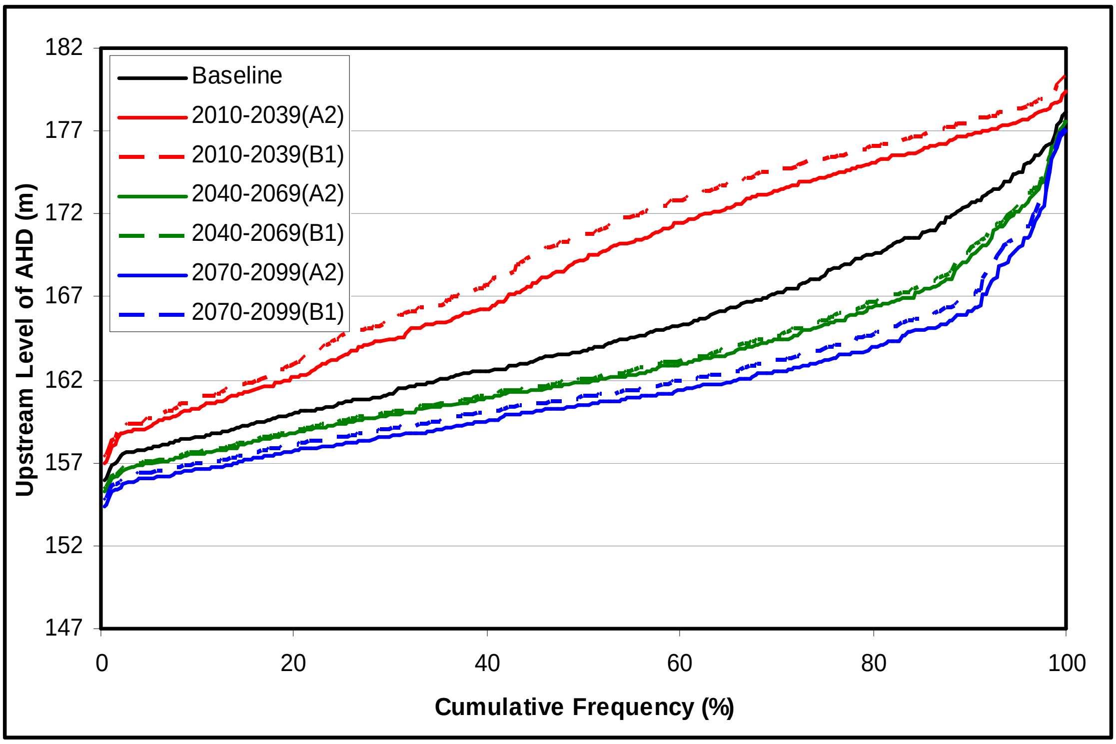


Figure B.4(e). Frequency curve of the AHD Upstream levels for scenario IV under the OPT policy.

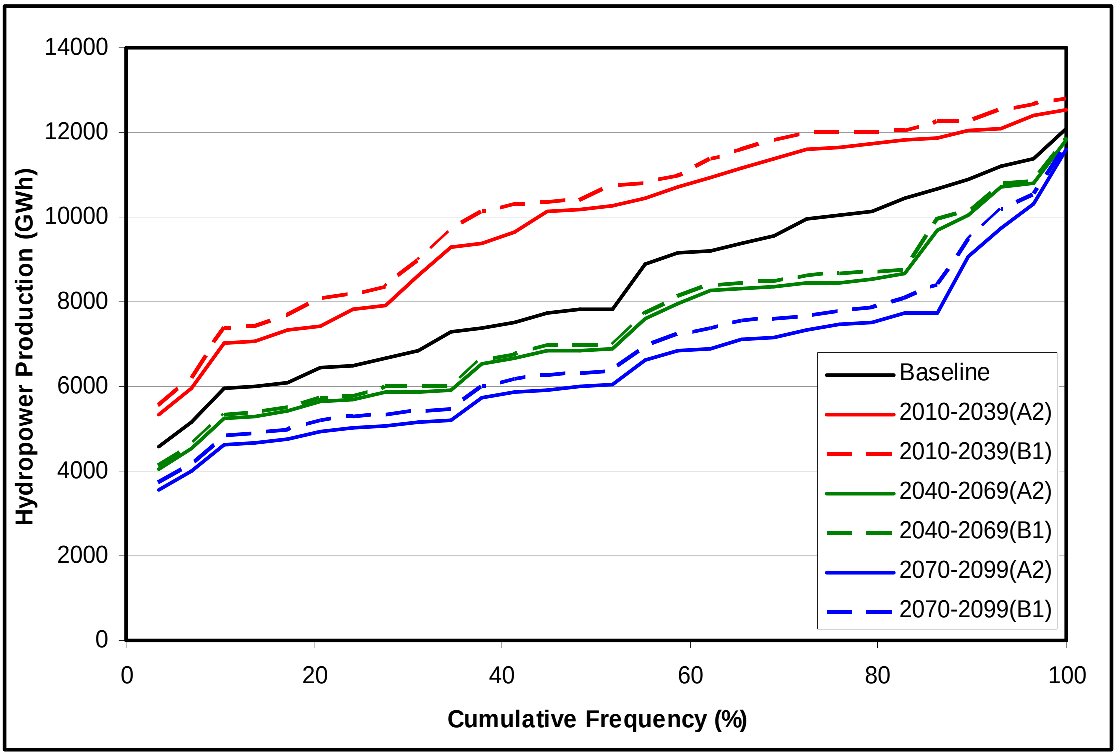


Figure B.4(f). Annual hydropower production frequency curve for scenario IV under the OPT policy.

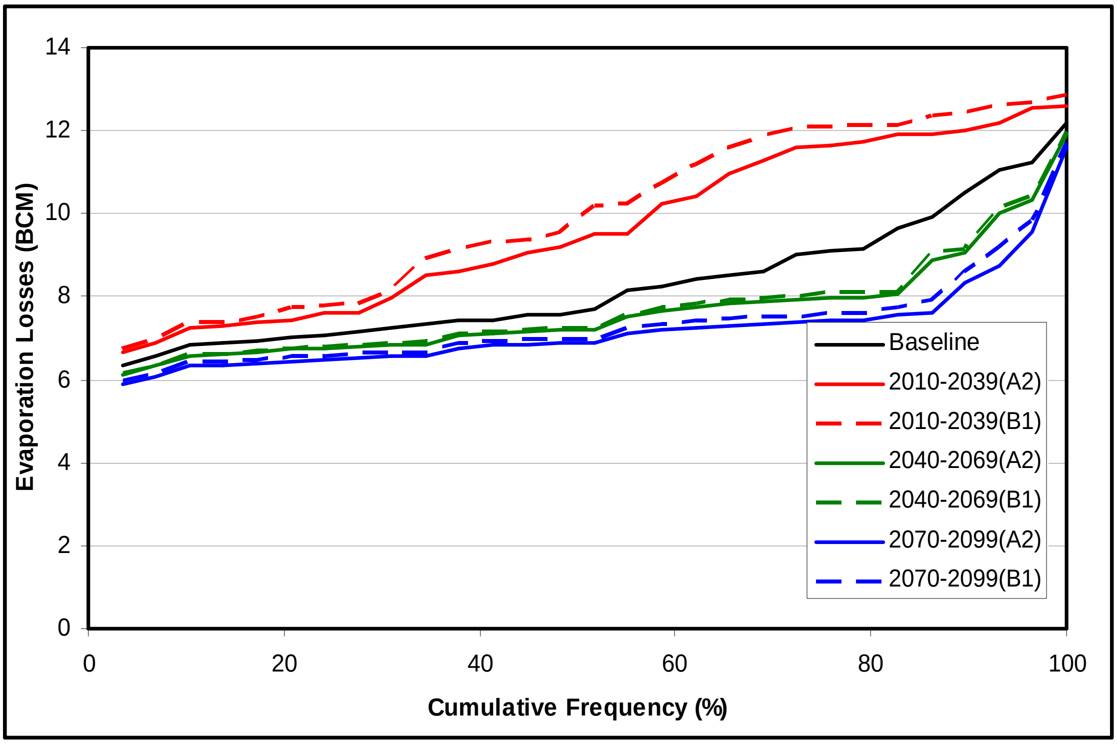


Figure B.4(g). Annual evaporation losses frequency curve for scenario IV under the OPT policy.

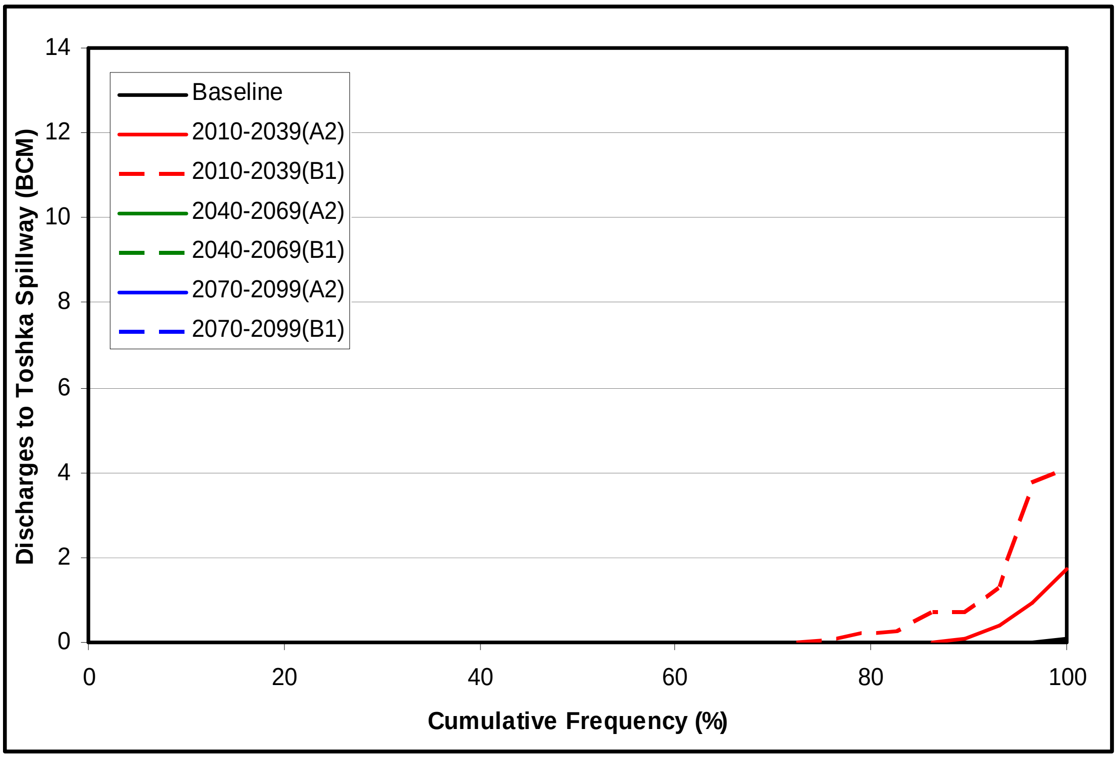


Figure B.4(h). Annual Toshka spillway discharges frequency curve for scenario IV under the OPT policy.

Scenario	Period	Emission scenario	Optimal Operation Rule (OPT)	Current Operation Rule (CUR)	Changes (OPT-CUR)
			BCM	BCM	BCM
Scenario I	Baseline		8.49	11.64	-3.15
	Period I	A2	9.76	12.39	-2.63
		B1	10.06	12.48	-2.42
	Period II	A2	7.80	10.70	-2.89
		B1	7.88	10.89	-3.01
	Period III	A2	7.30	9.21	-1.91
		B1	7.48	9.89	-2.41
Scenario II	Baseline		8.43	11.66	-3.23
	Period I	A2	9.74	12.38	-2.64
		B1	10.06	12.50	-2.44
	Period II	A2	7.77	10.76	-2.99
		B1	7.84	10.90	-3.06
	Period III	A2	7.28	9.21	-1.93
		B1	7.45	9.78	-2.32
Scenario III	Baseline		8.39	11.60	-3.22
	Period I	A2	9.71	12.37	-2.66
		B1	10.04	12.51	-2.47
	Period II	A2	7.74	10.76	-3.02
		B1	7.81	10.90	-3.09
	Period III	A2	7.26	9.13	-1.86
		B1	7.43	9.86	-2.44
Scenario IV	Baseline		8.31	11.71	-3.40
	Period I	A2	9.67	12.44	-2.77
		B1	10.05	12.54	-2.49
	Period II	A2	7.69	10.75	-3.06
		B1	7.76	10.90	-3.15
	Period III	A2	7.23	9.08	-1.85
		B1	7.39	9.81	-2.43

Table B.1. Comparison of annual average evaporation losses for optimal operation rule (OPT) and current operation rule (CUR).

Scenario	Period	Emission scenario	Optimal Operation Rule (OPT)	Current Operation Rule (CUR)	Changes (OPT-CUR)
			BCM	BCM	BCM
Scenario I	Baseline		0.08	3.28	-3.21
	Period I	A2	0.96	3.39	-2.43
		B1	1.26	3.52	-2.26
	Period II	A2	0.00	1.35	-1.35
		B1	0.00	1.55	-1.55
	Period III	A2	0.00	0.30	-0.30
		B1	0.00	0.59	-0.59
Scenario II	Baseline		0.08	3.04	-2.96
	Period I	A2	0.89	3.22	-2.32
		B1	1.28	3.54	-2.26
	Period II	A2	0.00	1.36	-1.36
		B1	0.00	1.48	-1.48
	Period III	A2	0.00	0.23	-0.23
		B1	0.00	0.48	-0.48
Scenario III	Baseline		0.08	3.00	-2.92
	Period I	A2	0.83	3.29	-2.46
		B1	1.28	3.54	-2.26
	Period II	A2	0.00	1.32	-1.32
		B1	0.00	1.43	-1.43
	Period III	A2	0.00	0.17	-0.17
		B1	0.00	0.56	-0.56
Scenario IV	Baseline		0.09	2.98	-2.90
	Period I	A2	0.80	3.29	-2.49
		B1	1.39	3.41	-2.02
	Period II	A2	0.00	1.23	-1.23
		B1	0.00	1.37	-1.37
	Period III	A2	0.00	0.19	-0.19
		B1	0.00	0.57	-0.57

Table B.2. Comparison of annual average discharges to Toshka spillway for optimal operation rule (OPT) and current operation rule (CUR).

Scenario	Period	Emission scenario	Optimal Operation Rule (OPT) BCM	Current Operation Rule (CUR) BCM	Changes (OPT-CUR) BCM
Scenario I	Baseline		62.23	56.00	6.23
	Period I	A2	68.03	61.56	6.46
		B1	69.44	63.33	6.10
	Period II	A2	57.58	52.70	4.87
		B1	58.17	53.20	4.97
	Period III	A2	52.68	50.42	2.25
		B1	54.53	51.32	3.21
Scenario II	Baseline		64.19	57.91	6.28
	Period I	A2	70.17	63.79	6.39
		B1	71.61	65.47	6.14
	Period II	A2	59.36	54.64	4.72
		B1	59.98	54.94	5.04
	Period III	A2	54.29	52.01	2.28
		B1	56.21	53.13	3.08
Scenario III	Baseline		65.86	59.68	6.18
	Period I	A2	72.02	65.65	6.37
		B1	73.47	67.28	6.19
	Period II	A2	60.88	56.11	4.77
		B1	61.52	56.43	5.09
	Period III	A2	55.67	53.46	2.21
		B1	57.64	54.38	3.26
Scenario IV	Baseline		69.96	63.47	6.49
	Period I	A2	76.53	69.90	6.63
		B1	78.02	71.79	6.23
	Period II	A2	64.63	59.81	4.82
		B1	65.31	60.12	5.19
	Period III	A2	59.08	56.89	2.19
		B1	61.18	57.92	3.26

Table B.3. Comparison of annual average water supply releases for optimal operation rule (OPT) and current operation rule (CUR).

Scenario	Period	Emission scenario	Optimal Operation Rule (OPT) GWh	Current Operation Rule (CUR) GWh	Changes (OPT/CUR) %
Scenario I	Baseline		7451	7570	98
	Period I	A2	8734	8565	102
		B1	9046	8850	102
	Period II	A2	6590	6856	96
		B1	6693	6982	96
	Period III	A2	5797	6079	95
		B1	6088	6412	95
Scenario II	Baseline		7682	7875	98
	Period I	A2	9018	8913	101
		B1	9344	9193	102
	Period II	A2	6798	7158	95
		B1	6904	7243	95
	Period III	A2	5983	6297	95
		B1	6282	6626	95
Scenario III	Baseline		7876	8116	97
	Period I	A2	9257	9201	101
		B1	9597	9477	101
	Period II	A2	6973	7371	95
		B1	7081	7460	95
	Period III	A2	6140	6463	95
		B1	6445	6833	94
Scenario IV	Baseline		8368	8722	96
	Period I	A2	9856	9878	100
		B1	10229	10181	100
	Period II	A2	7414	7903	94
		B1	7528	8000	94
	Period III	A2	6534	6904	95
		B1	6856	7306	94

Table B.4. Comparison of annual average hydropower production for optimal operation rule (OPT) and current operation rule (CUR).

APPENDIX C: LEVELS VARIATION

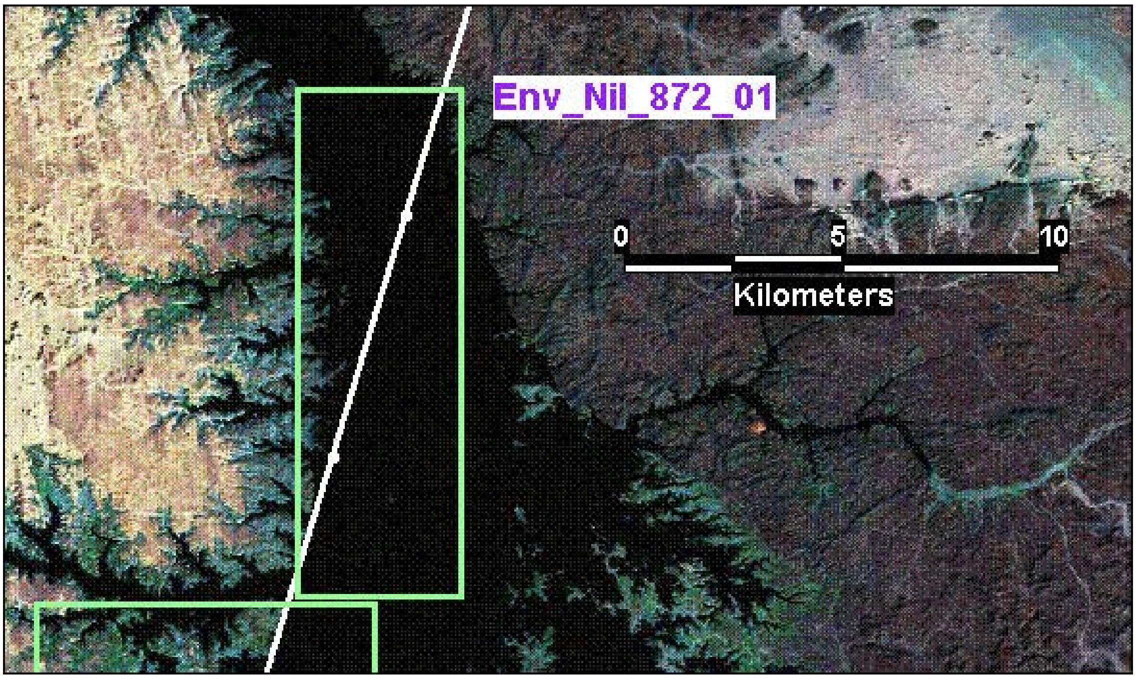


Figure C.1. Satellit altimerty track (Env_Nil_872_01) over the AHDR [LEGOS, 2009].

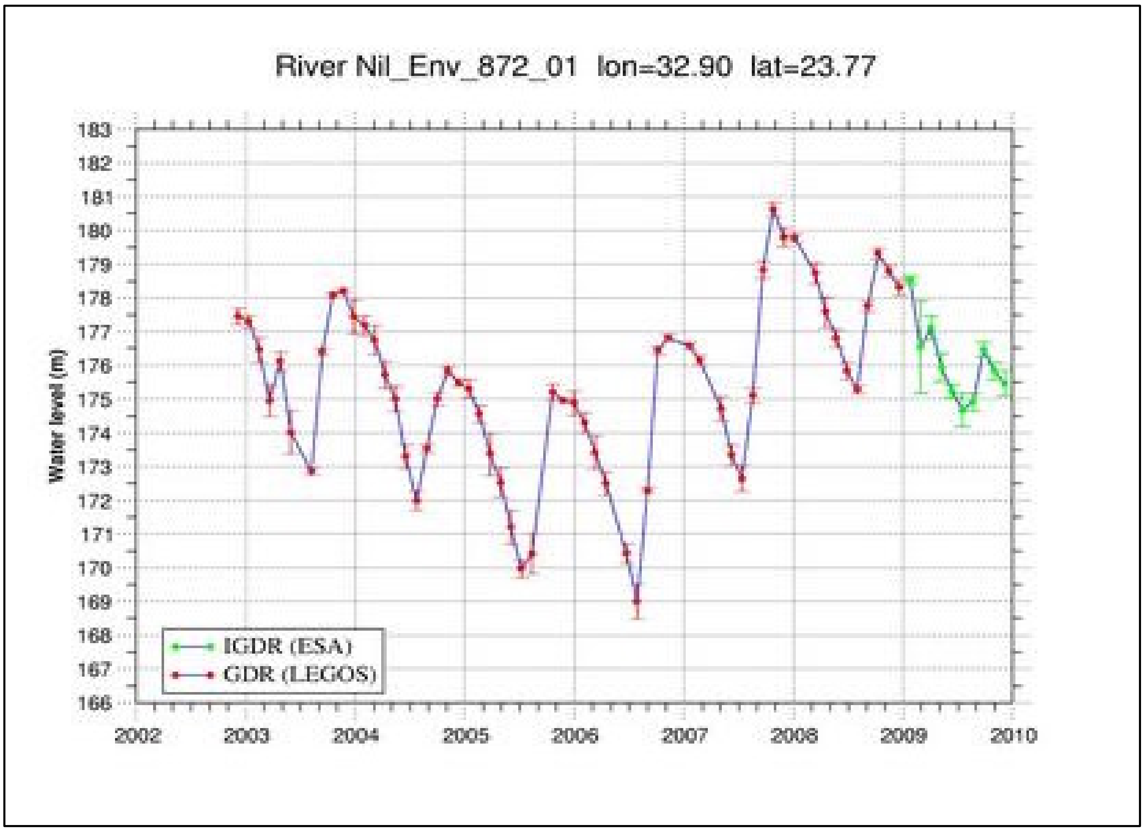


Figure C.2. Water levels at track (Env_Nil_872_01) [LEGOS, 2009].

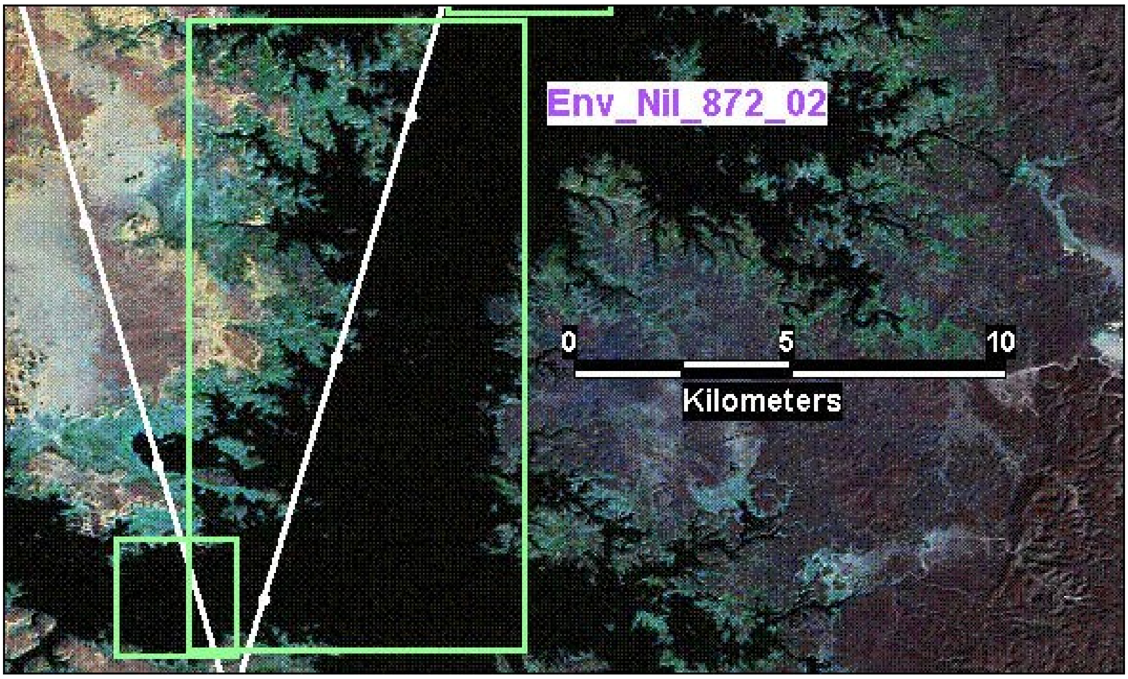


Figure C.3. Satellit altimerty track (Env_Nil_872_02) over the AHDR [LEGOS, 2009].

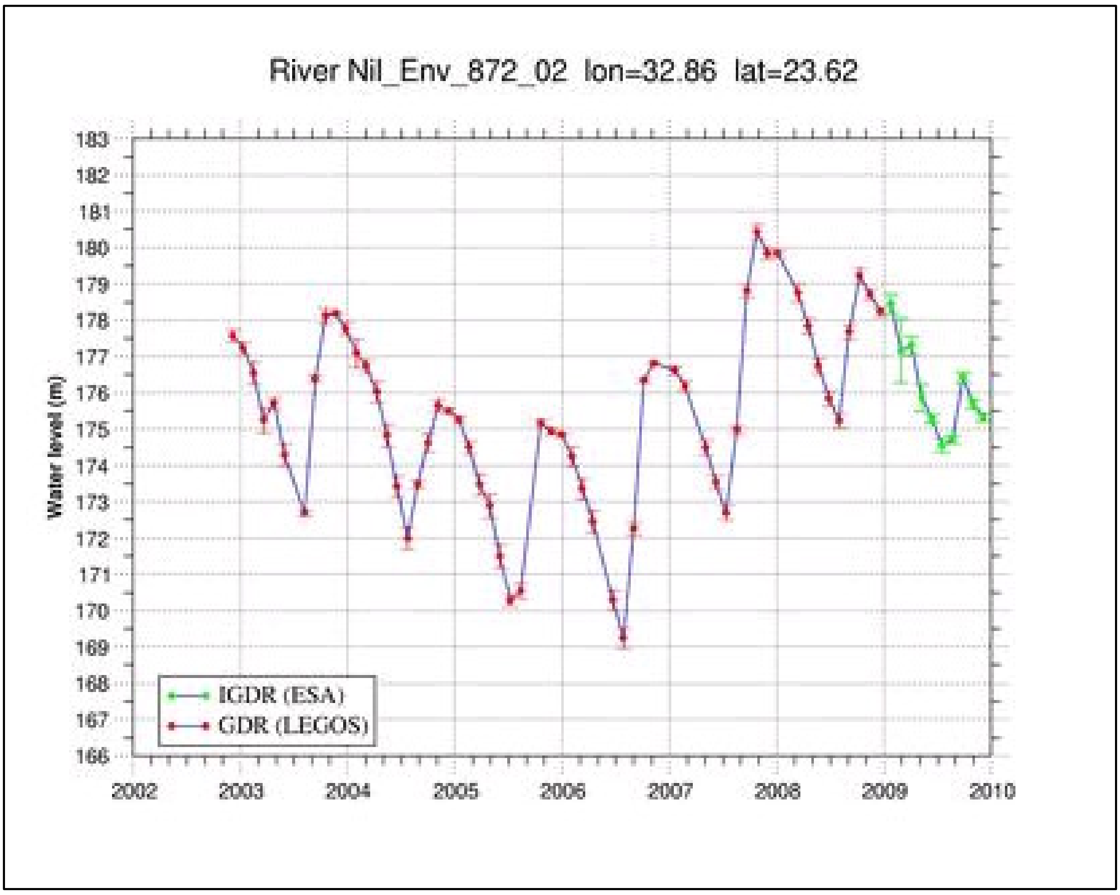


Figure C.4. Water levels at track (Env_Nil_872_02) [LEGOS, 2009].

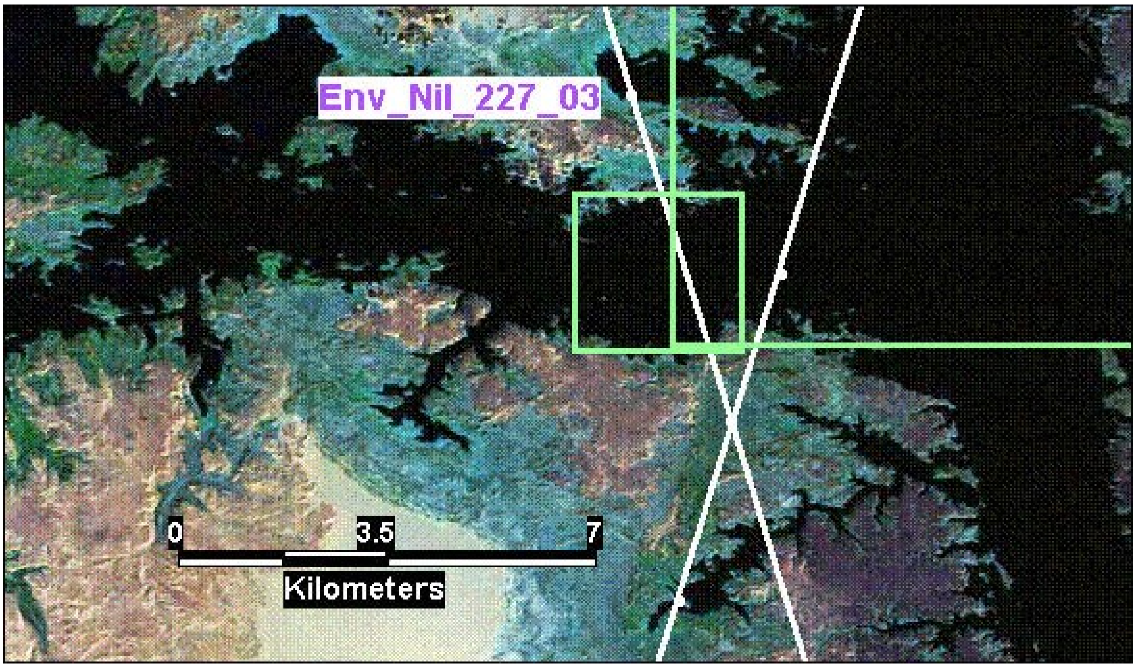


Figure C.5. Satellit altimerty track (Env_Nil_227_03) over the AHDR [LEGOS, 2009].

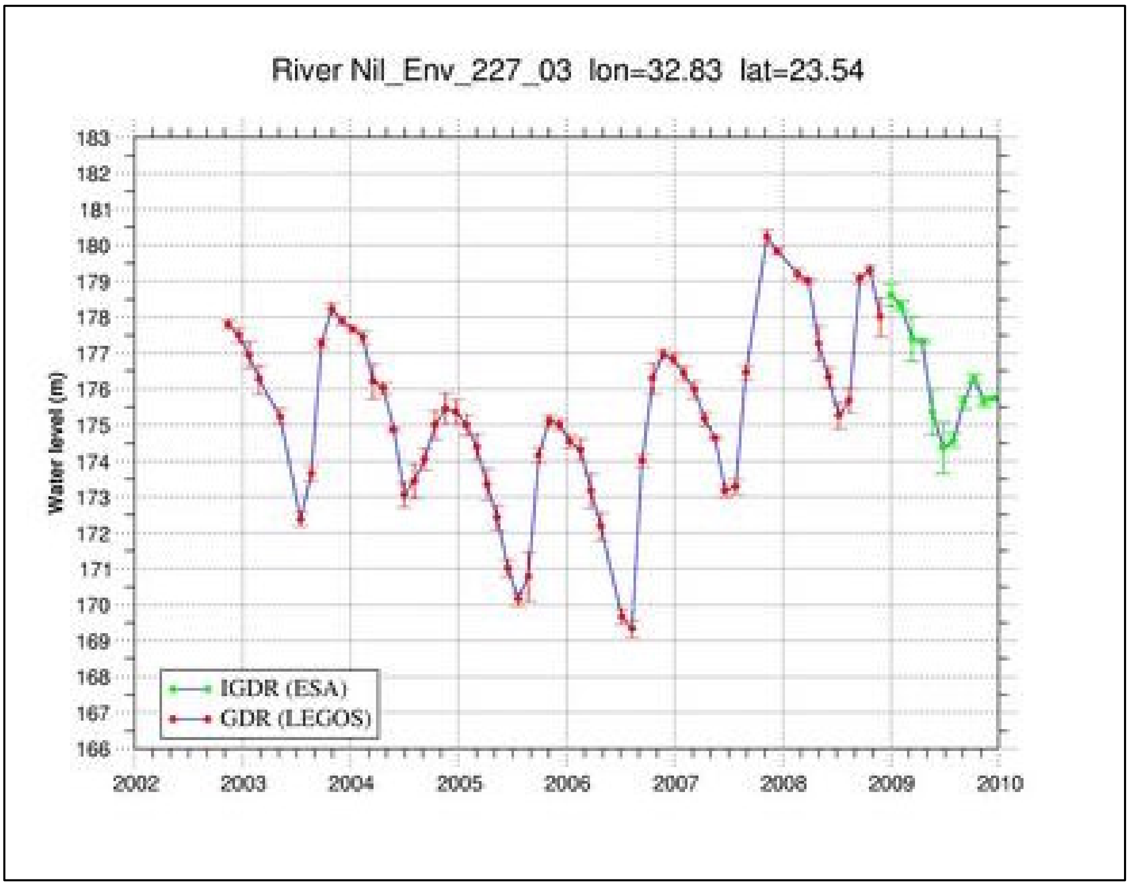


Figure C.6. Water levels at track (Env_Nil_227_03) [LEGOS, 2009].

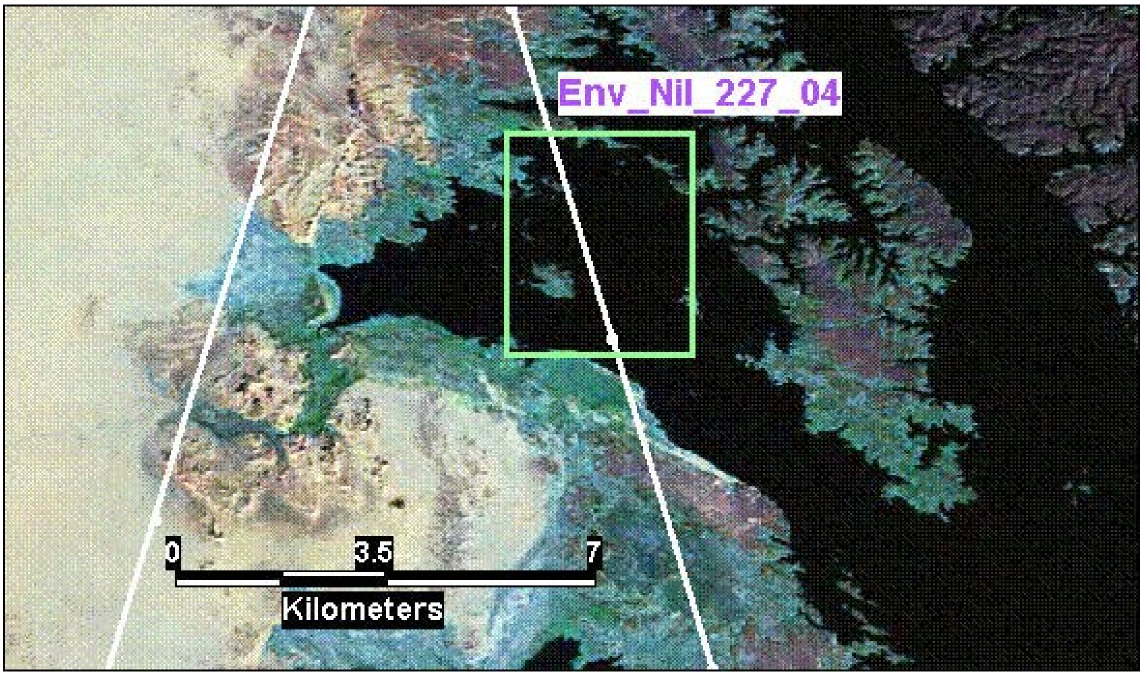


Figure C.7. Satellit altimerty track (Env_Nil_227_04) over the AHDR [LEGOS, 2009].

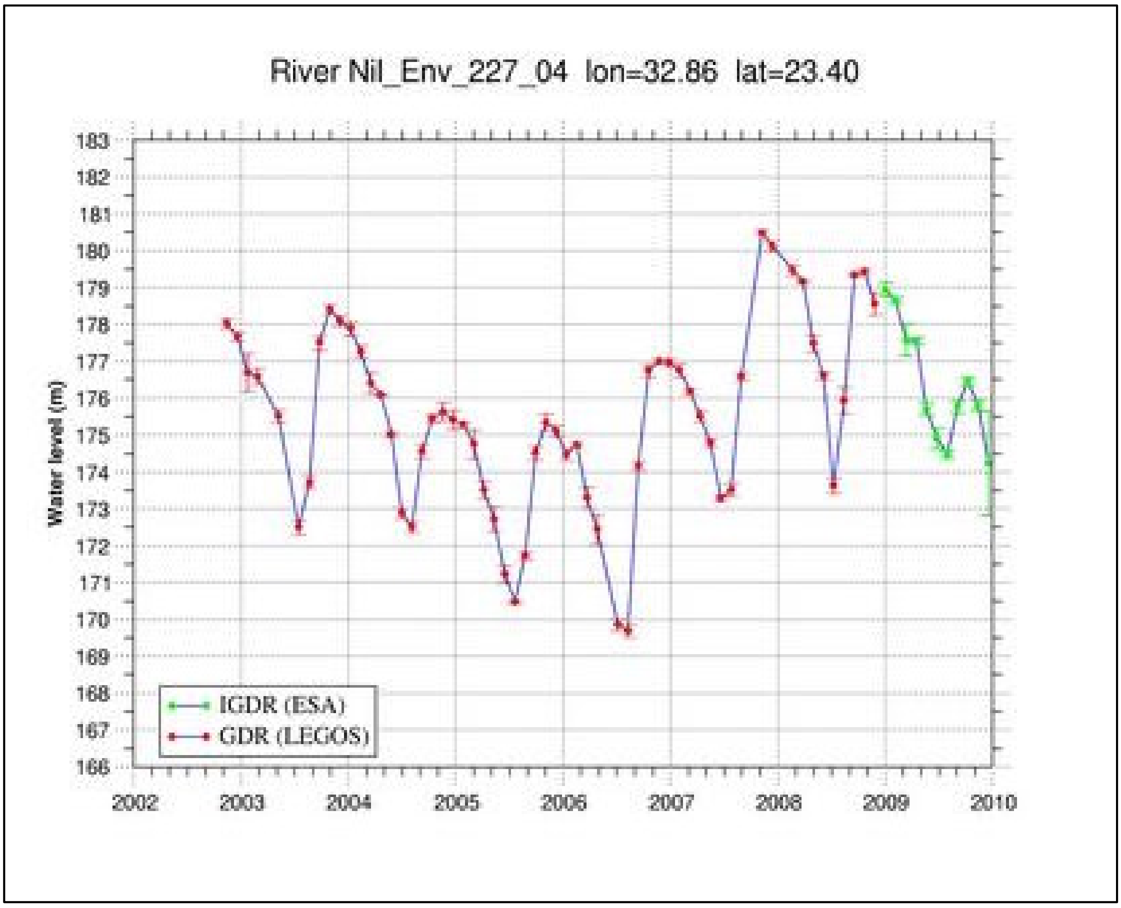


Figure C.8. Water levels at track (Env_Nil_227_04)[LEGOS, 2009].

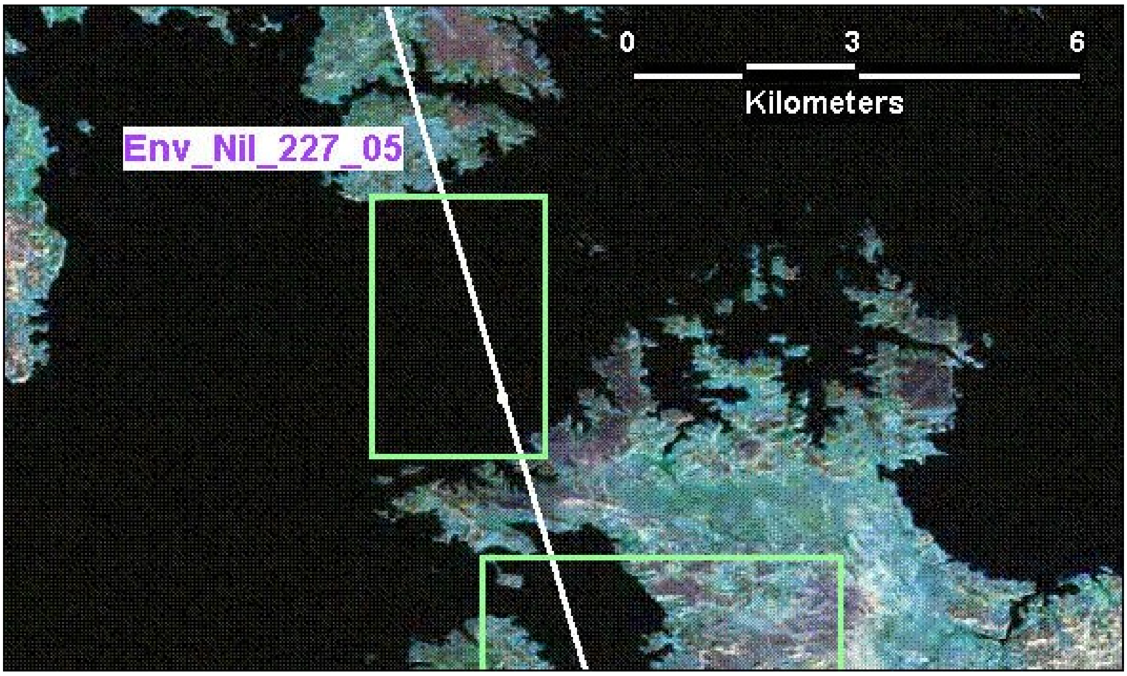


Figure C.9. Satellit altimerty track (Env_Nil_227_05) over the AHDR [LEGOS, 2009].

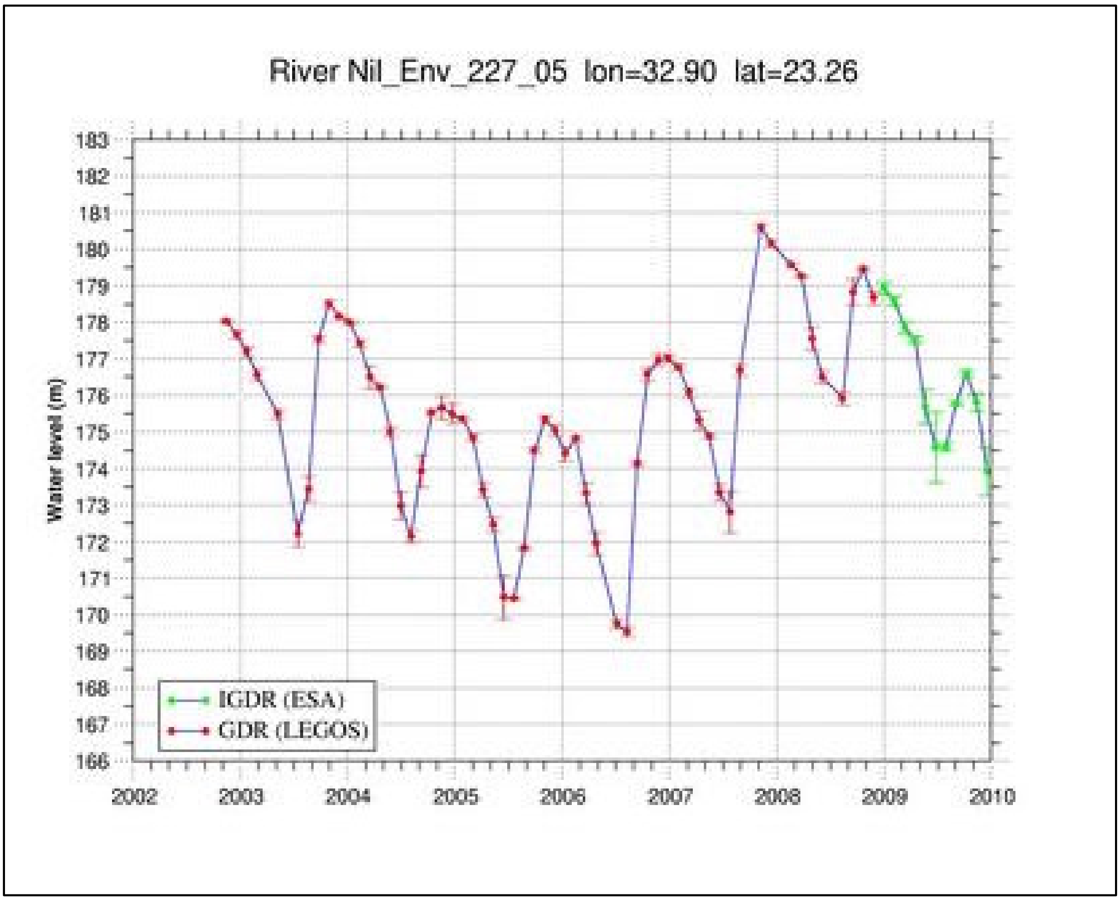


Figure C.10. Water levels at track (Env_Nil_227_05) [LEGOS, 2009].

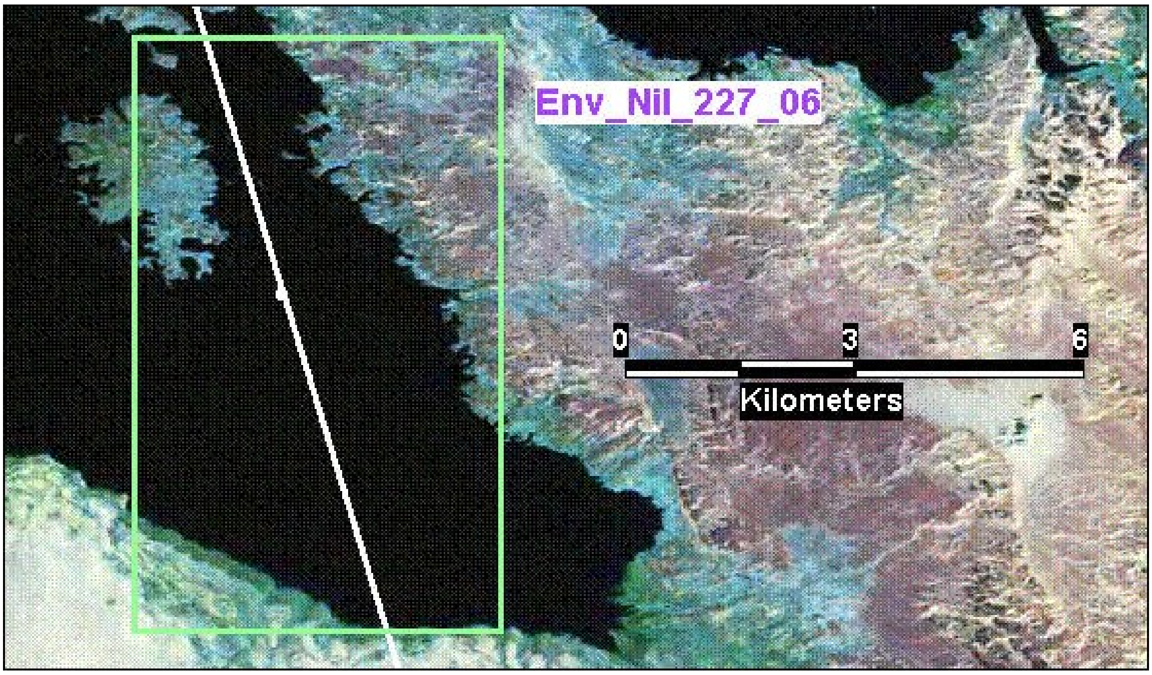


Figure C.11. Satellit altimerty track (Env_Nil_227_06) over the AHDR [LEGOS, 2009].

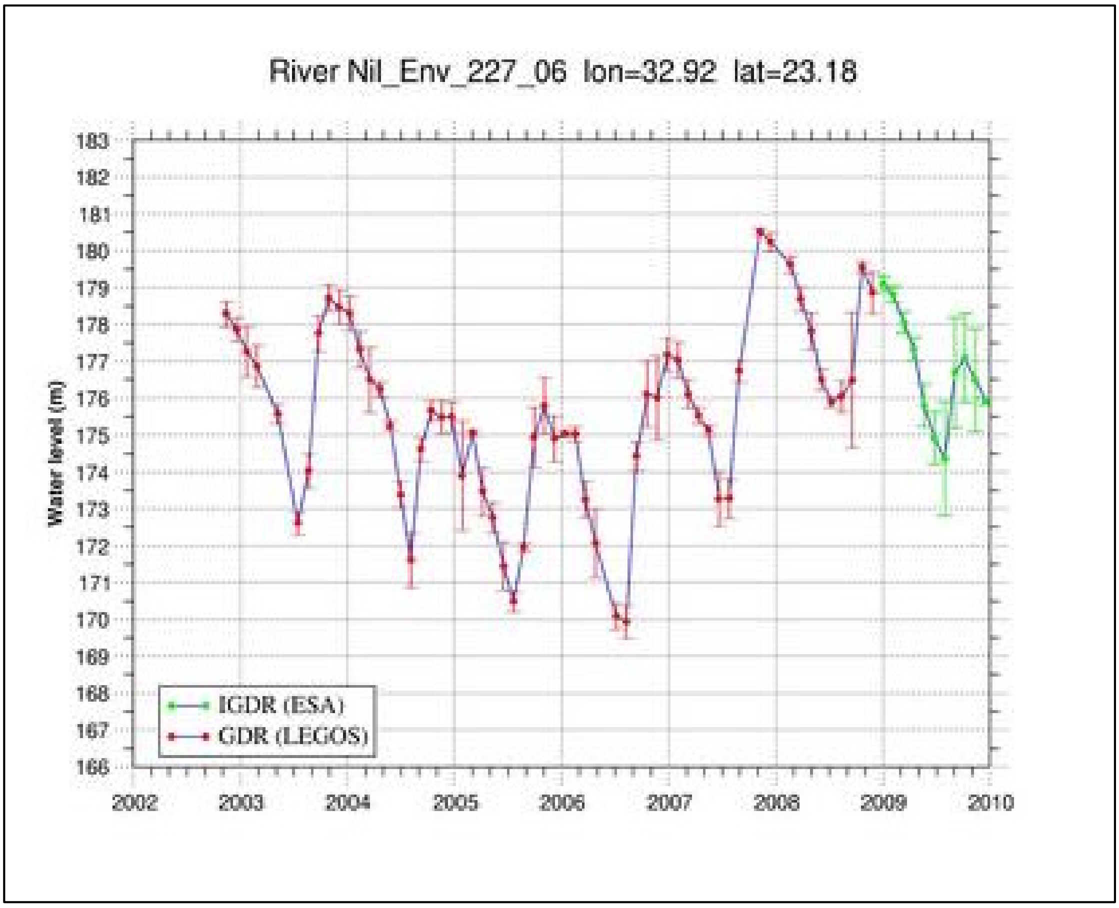


Figure C.12. Water levels at track (Env_Nil_227_06)[LEGOS, 2009].

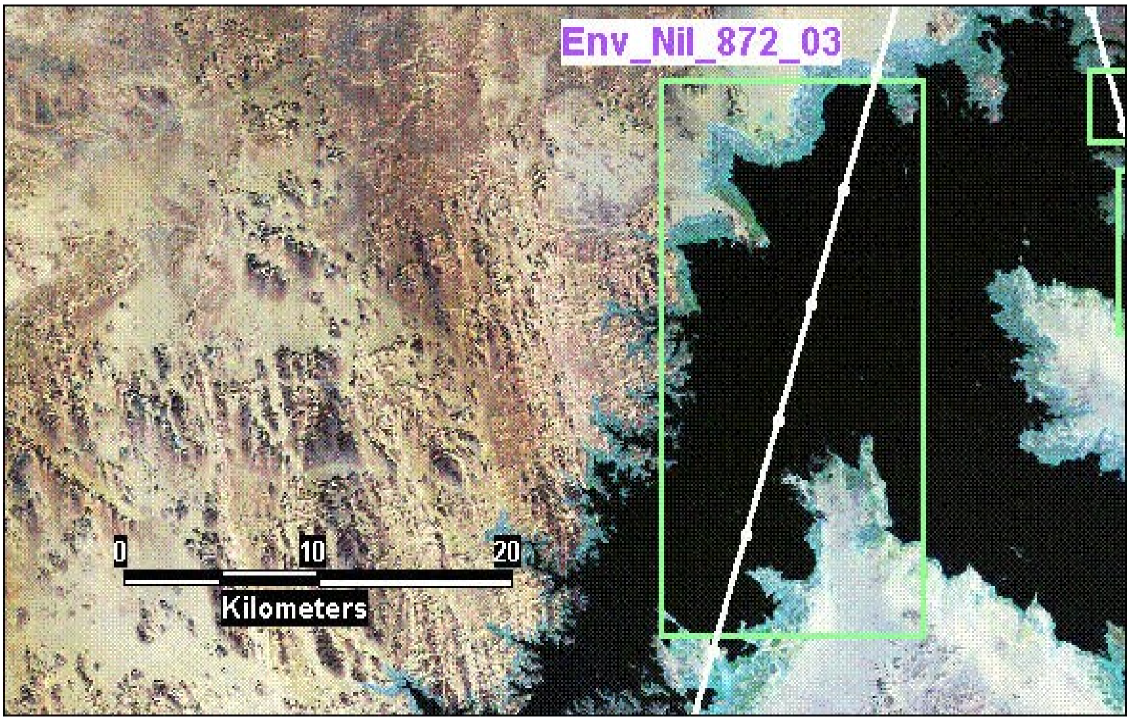


Figure C.13. Satellit altimerty track (Env_Nil_872_03) over the AHDR [LEGOS, 2009].

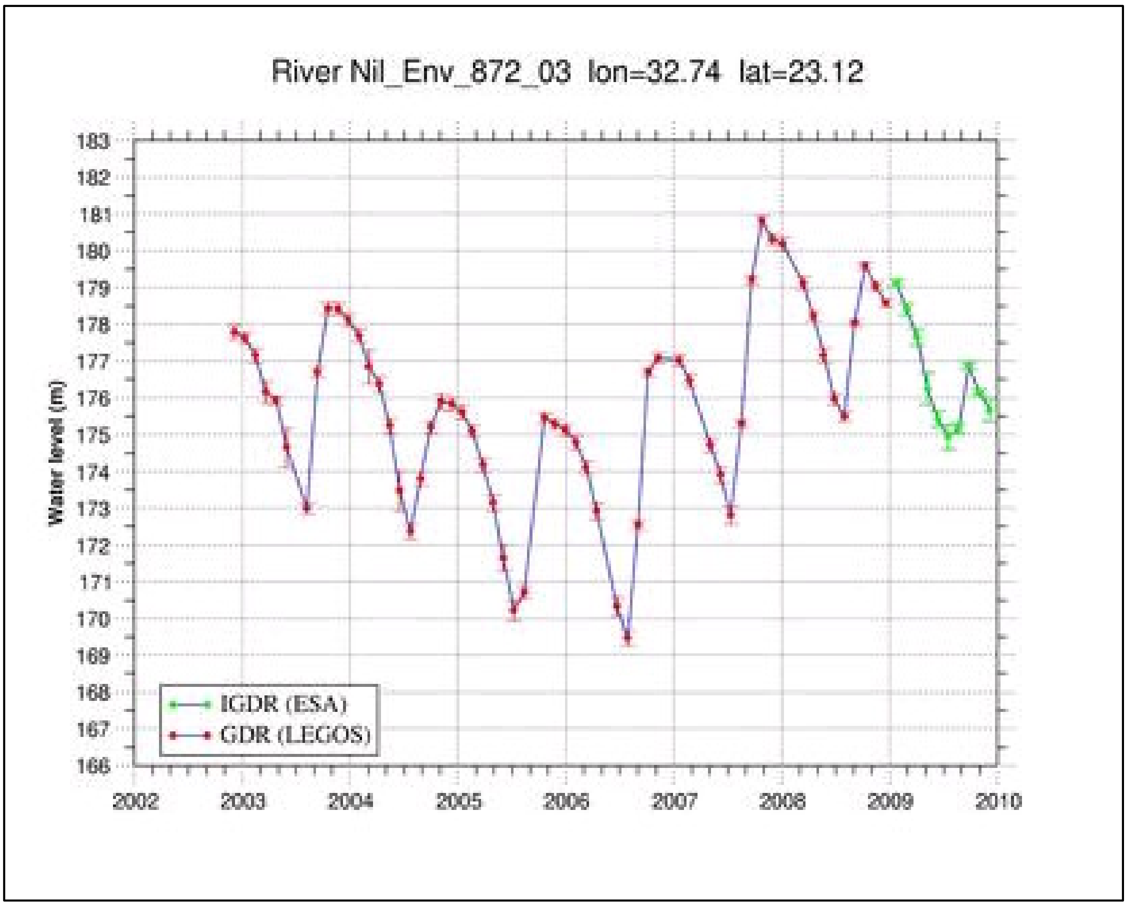


Figure C.14. Water levels at track (Env_Nil_872_03) [LEGOS, 2009].

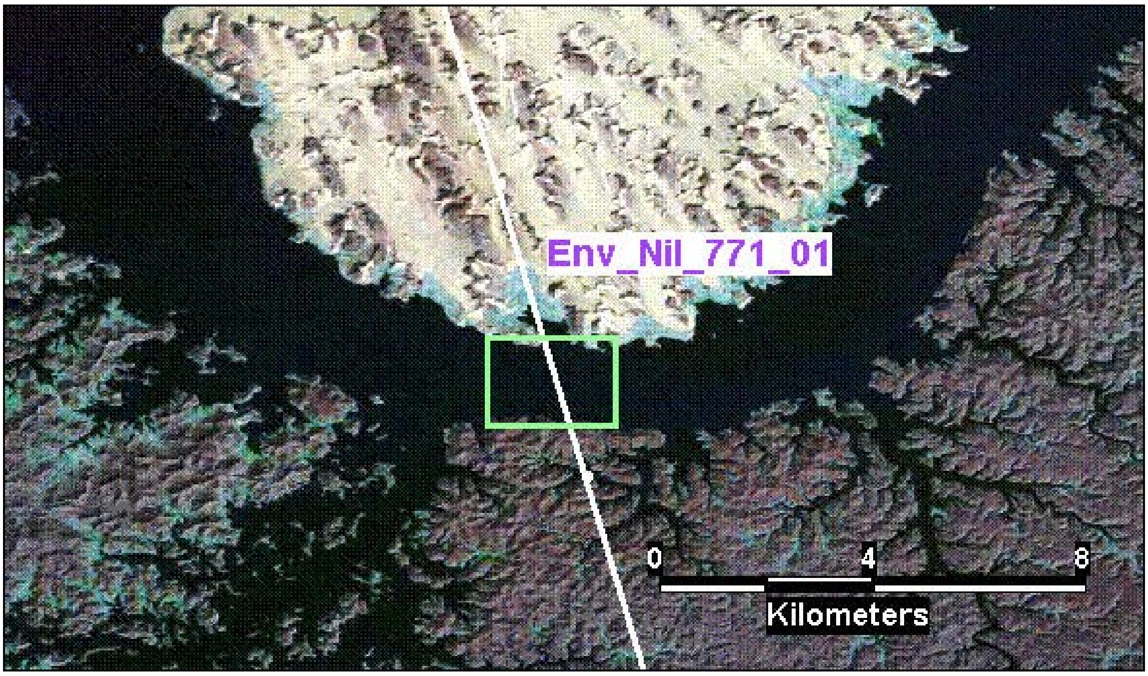


Figure C.15. Satellit altimerty track (Env_Nil_771_01) over the AHDR [LEGOS, 2009].

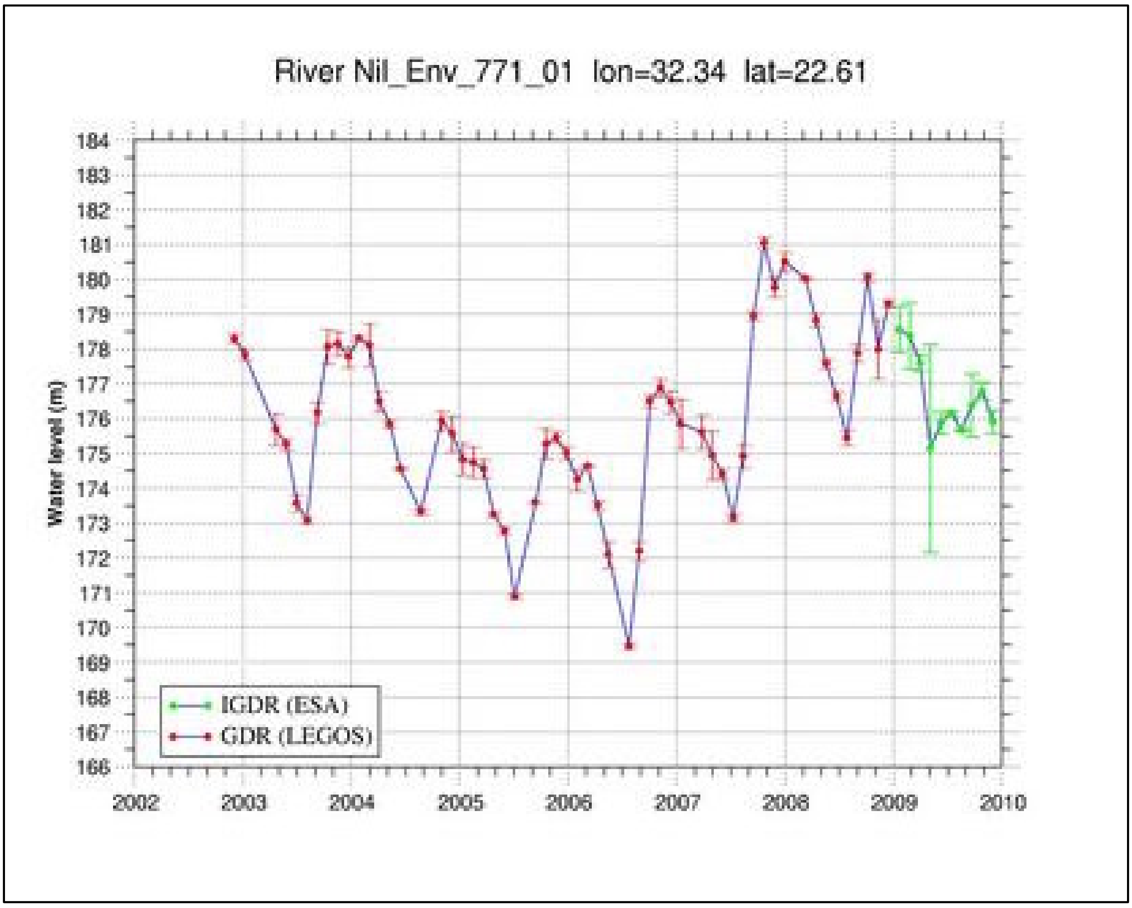


Figure C.16. Water levels at track (Env_Nil_771_01)[LEGOS, 2009].

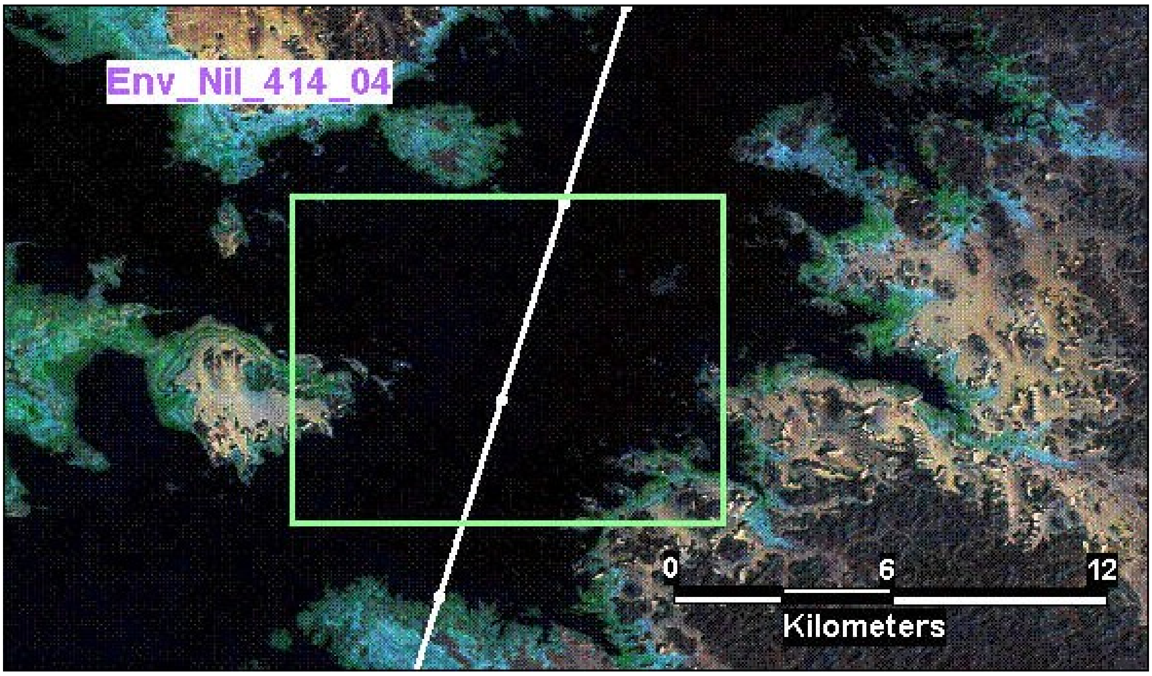


Figure C.17. Satellit altimerty track (Env_Nil_414_04) over the AHDR [LEGOS, 2009].

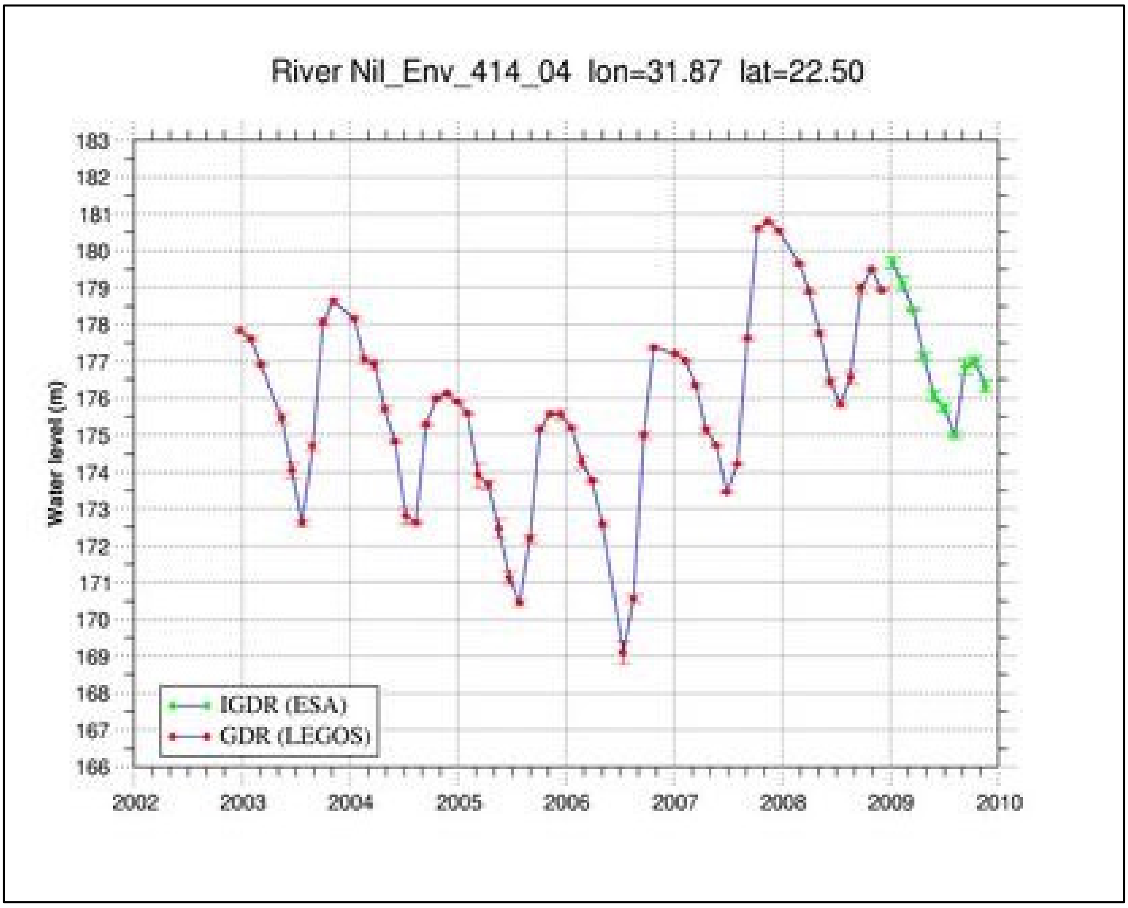


Figure C.18. Water levels at track (Env_Nil_414_04) [LEGOS, 2009].

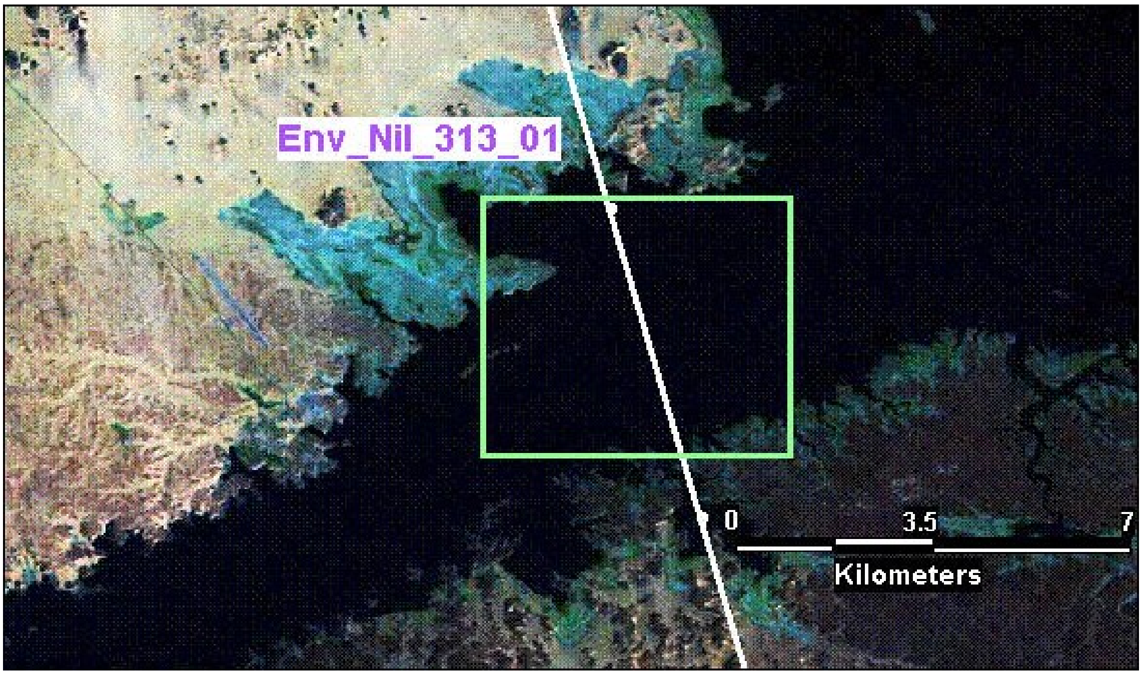


Figure C.19. Satellit altimerty track (Env_Nil_313_01) over the AHDR [LEGOS, 2009].

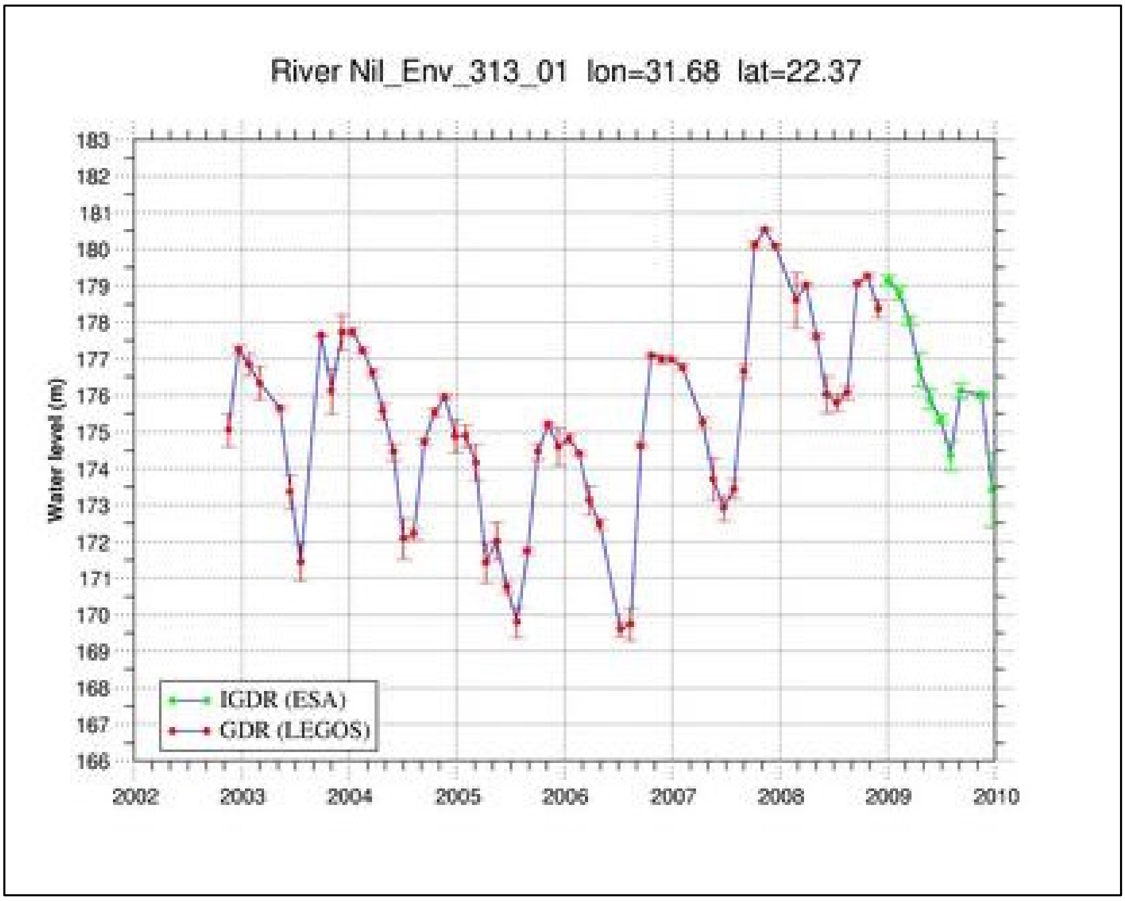


Figure C.20. Water levels at track (Env_Nil_313_01) [LEGOS, 2009].

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Professional Appointments

Doctoral student, Institute of Hydraulic and Water Resources Engineering, Darmstadt University of Technology, Germany, 2007 – 2010.
Assistant lecture, Civil Engineering Department, Faculty of Engineering, Al-Azhar University, Egypt, 2005– 2006.
Demonstrator, Civil Engineering Department, Faculty of Engineering, Al-Azhar University, Egypt, 2001– 2005.

Education

Ph.D., Civil Engineering and Geodesy, Darmstadt University of Technology, Germany, 2010.
M.Sc., Civil Engineering, Al-Azhar University, Egypt, 2005.
B.Sc. (Excellent with honor), Civil Engineering, Al-Azhar University, Egypt, 2000.

Languages

English (understanding, speaking, reading, writing, listening): Very Good
German (Grundstufe III Certificate, Goethe Institute in Cairo).

Computer Skills

BlueM , Matlab 7, SAP 2000, Autocad 2006 and Microsoft Offic.

Research Interests

Adaptive Water Management; reservoirs simulation and optimization; streamflow forecasting; water resources management; global change impact assessments for water resources; uncertainty characterization.

Publications & Conferences

- Amir Mobasher and Manfred Ostrowski, "Improving Dam Safety through Re-Operation of Multi-Purpose Reservoir," " The Symposium on DAM SAFETY - Sustainability in a Changing Environment", 21st to the 24th of September 2010, Innsbruck, Austria.
- Amir Mobasher and Manfred Ostrowski, "Dynamic Reservoir Operation for Aswan High Dam Reservoir to Increase Nile Water Availability," "Wasserbau in Bewegung - von der Statik zur Dynamik", Symposium 1 - 3 July 2010, TU Muenchen, Wallgau, Germany.
- Amir Mobasher and Manfred Ostrowski, "An Assessment of Reservoir Operation Sensitivity to Climate Changes – The Case Study of Lake Nasser," The First European Congress of the IAHR, 4th – 6th May 2010, Edinburgh, UK.
- Amir Mobasher, "Using The Modelling System BlueM for The Case Of Aswan High Dam Reservoir," Darmstaedter Ingenieurkongress - Bau und Umwelt, 14. und 15. September 2009, Darmstadt, Germany.
- Amir Mobasher and Manfred Ostrowski, "Adaptive Reservoir Operation Strategies under Changing Boundary Conditions – The case of Aswan High Dam Reservoir," Day of Hydrology, Ecology-Centre of Christian-Albrechts University of Kiel, 26 – 27 March 2009, Kiel, Germany.
- The International conference "Nature-oriented Flood Damage Prevention- Planning, Evaluation and Communication", 16 – 18 April, 2008, Darmstadt, Germany.

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